A TREATISE

ON

MASONRY CONSTRUCTION.

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PREFACE.

THE present volume is an outgrowth of the needs of the author's own class-room. The matter is essentially that presented to his classes for a number of years past, a considerable part having been used in the form of a blue-print manuscript text-book. It is now published for the greater convenience of his own students, and with the hope that it may be useful to others. The author knows of no work which treats of any considerable part of the field covered by this volume. Nearly all of the matter is believed to be entirely new.

The object has been to develop principles and methods and to give such examples as illustrate them, rather than to accumulate details or to describe individual structures. The underlying principles of ordinary practice are explained; and, where needed, ways are pointed out whereby it may be improved. The common theories are compared with the results of actual practice; and only those are recommended which have been verified by experiments or experience, since true theory and good practice are always in accord. The author has had the benefit of suggestions and advice from practical masons and engineers, and believes that the information here presented is reliable, and that the examples cited represent good practice. The general prices are the average of a large number actually paid; and the special prices are representative. The structures illustrated are actual ones. The accredited illustrations are from well-authenticated copies of working drawings, and are presented without any modification whatever; while those not accredited are representative of practice so common that a single name could not properly be attached.

In the preparation of the book the endeavor has been to observe a logical order and a due proportion between different parts. Great care has been taken in classifying and arranging the matter. It will be helpful to the reader to notice that the volume is divided successively into parts, chapters, articles, sections having small-capital black-face side-heads, sections having lower-case black-face side-heads, sections having lower-case italic side-heads, and sections having simply the serial number. In some cases the major subdivis-

ions of the sections are indicated by small numerals. The constant aim has been to present the subject clearly and concisely.

Every precaution has been taken to present the work in a form for convenient practical use and ready reference. Numerous cross references are given by section number; and whenever a figure or a table is mentioned, the citation is accompanied by the number of the page on which it may be found. The table of contents shows the general scope of the book; the running title assists in finding the different parts; and a very full index makes everything in the book easy of access. There are also a number of helps for the student, which the experienced teacher will not fail to recognize and appreciate.

Although the book has been specially arranged for engineering and architectural students, it is hoped that the information concerning the strengths of the materials, the data for facilitating the making of estimates, the plans, the tables of dimensions, and the costs of actual structures, will prove useful to the man of experience. Considering the large amount of practical details presented and the great difference in the methods employed by various constructors, it is probable that practical men will find much to criticise. The views here expressed are, however, the results of observation throughout the entire country, and of consultation and correspondence with many prominent and practical men, and represent average good practice. The experienced engineer may possibly also feel that some subjects should have been treated more fully; but it is neither wise nor possible to give in a single volume minute details. These belong to technical journals, proceedings of societies, and special reports of particular work.

No pains have been spared in verifying data and checking results. The tables of cubic contents have been computed by different processes by at least two persons, and to at least one more place than is recorded. Should any error, either of printer or author, be discovered—as is very possible in a work of so much detail, despite the great care used,—the writer will be greatly obliged by prompt notification of the same.

The author gratefully acknowledges his indebtedness to many engineers for advice and data, and to his former pupil and present co-laborer, Prof. A. N. Talbot, for many valuable suggestions.

CHAMPAIGN, ILL., July 9, 1889.

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MASONRY CONSTRUCTION.

INTRODUCTION.

UNDER this general head will be discussed the subjects relating to the use of stone and brick as employed by the engineer or architect in the construction of buildings, retaining walls, bridge piers, culverts, arches, etc., including the foundations for the same. For convenience, the subject will be divided as follows:

Part I. Description and Characteristics of the Materials.
Part II. Methods of Preparing and Using the Materials.
Part III. Foundations.
Part IV. Masonry Structures.

"The first cost of masonry should be its only cost. Though superstructures decay and drift away, though embankments should crumble and wash out, masonry should stand as one great mass of solid rock, firm and enduring."

—*Anonymous.*

PART I.

THE MATERIALS.

CHAPTER I.

NATURAL STONE.

ART. 1. REQUISITES FOR GOOD BUILDING STONE.

1. The qualities which are most important in stone used for construction are cheapness, durability, strength, and beauty.

2. CHEAPNESS. The primary factor which determines the value of a stone for structural purposes is its cheapness. The items which contribute to the cheapness of a stone are abundance, proximity of quarries to place of use, facility of transportation, and the ease with which it is quarried and worked.

The wide distribution and the great variety of good building stone in this country are such that suitable stone should everywhere be cheap. That such is not the case is probably due either to a lack of the development of home resources or to a lack of confidence in home products. The several State and Government geological surveys have done much to increase our knowledge of the building stones of this country.

The lack of confidence in home resources has very frequently caused stones of demonstrated good quality to be carried far and wide, and frequently to be laid down upon the outcropping ledges of material in every way their equal. The first stone house erected in San Francisco, for example, was built of stone brought from China; and at the present day the granites mostly employed there are brought from New England or from Scotland. Yet there are no stones in our country more to be recommended than the California granites. Some of the prominent public and private buildings in Cincinnati are constructed of stone that was carried by water and

railway a distance of about 1500 miles. Within 150 miles of Cincinnati, in the sub-carboniferous limestone district of Kentucky, there are very extensive deposits of dolomitic limestone that afford a beautiful building stone, which can be quarried at no more expense than that of the granite of Maine. Moreover, this dolomite is easily carved, and requires not more than one third the labor to give it a surface that is needed by granite. Experience has shown that the endurance of this stone under the influence of weather is very great; yet because it has lacked authoritative indorsement there has been little market for it, and lack of confidence in it has led to the transportation half-way across the continent of a stone little, if any, superior to it.

Development of local resources follows in the wake of good information concerning them, for the lack of confidence in home products can not be attributed to prejudice.

The facility with which a stone may be quarried and worked is an element affecting cheapness. To be cheaply worked, a stone must not only be as soft as durability will allow, but it should have no flaws, knots, or hard crystals.

3. **Durability.** Next in importance after cheapness is durability. Rock is supposed to be the type of all that is unchangeable and lasting; but the truth is that, unless a stone is suited to the conditions in which it is placed, there are few substances more liable to decay and utter failure. The durability of stone is a subject upon which there is very little reliable knowledge. The question of endurance under the action of weather and other forces can not be readily determined. The external aspect of the stone may fail to give any clue to it; nor can all the tests we yet know determine to a certainty, in the laboratory, just how a given rock will withstand the effect of our variable climate and the gases of our cities. If our land were what is known as a rainless country, and if the temperature were uniform throughout the year, the selection of a durable building stone would be much simplified. The cities of northern Europe are full of failures in the stones of important structures. The most costly building erected in modern times, perhaps the most costly edifice reared since the Great Pyramid,—the Parliament House in London,—was built of a stone taken on the recommendation of a committee representing the best scientific and technical skill of Great Britain. The stone selected was submitted

to various tests, but the corroding influence of a London atmosphere was overlooked. The great structure was built, and now it seems questionable whether it can be made to endure as long as a timber building would stand, so great is the effect of the gases of the atmosphere upon the stone. This is only one of the numerous instances that might be cited in which a neglect to consider the climatic conditions of a particular locality in selecting a building material has proved disastrous.

"The great difference which may exist in the durability of stones of the same kind, presenting little difference in appearance, is strikingly exemplified at Oxford, England, where Christ Church Cathedral, built in the twelfth or thirteenth century of oölite from a quarry about fifteen miles away, is in good preservation, while many colleges only two or three centuries old, built also of oölite from a quarry in the neighborhood of Oxford, are rapidly crumbling to pieces." *

4. **STRENGTH.** The strength of stone is in some instances a cardinal quality, as when it is to form piers or columns to support great weights, or capstones that span considerable intervals. It is also an indispensable attribute of stone that is to be exposed to mechanical violence or unusual wear, as in steps, lintels, door-jambs, etc.

5. **BEAUTY.** This element is of more importance to the architect than to the engineer; and yet the latter can not afford to neglect entirely the element of beauty in the design of his most utilitarian structures. The stone should have a durable and pleasing color.

ART. 2. TESTS OF THE QUALITY OF BUILDING STONES.

6. As a general rule, the densest, hardest, and most uniform stone will most nearly meet the preceding requisites for a good building stone. The fitness of stone for structural purposes can be determined approximately by examining a fresh fracture. It should be bright, clean, and sharp, without loose grains, and free from any dull, earthy appearance. The stone should contain no "drys," *i.e.*, seams containing material not thoroughly cemented together, nor "crow-foots," *i.e.*, veins containing dark-colored, uncemented material.

* Rankine's Civil Engineering, p. 362.

The quality is usually tested by determining its strength,—particularly its resistance to crushing. This form of test is resorted to, not because of the predominating importance of the quality of strength, nor because of any known relation which exists between strength and durability, but because of the greater facility with which the strength can be determined.

STRENGTH.

7. Crushing Strength. The crushing strength of a stone is generally tested by applying measured force to cubes until they are crushed. Usually, when simply the strength of stone is referred to, the crushing strength is intended. The results for the crushing strength vary greatly with the details of the experiments. Several points, which should not be neglected either in planning a series of experiments or in using the results obtained by experiment, will be taken up separately, although they are not entirely independent.

8. Form of Fracture. Homogeneous stones in small cubes appear in all cases to break as shown in Fig. 1. The forms of the fragments *a* and *b* are, approximately, either conical or pyramidal. The more or less disk-shaped pieces *c* and *d* are detached from the sides of the cube with a kind of explosion. In the angles *e* and *f*, the stone is generally found crushed and ground into powder. A homogeneous stone is possibly imaginary, but some of the artificial stones described in a succeeding chapter are nearly, if not quite, homogeneous.

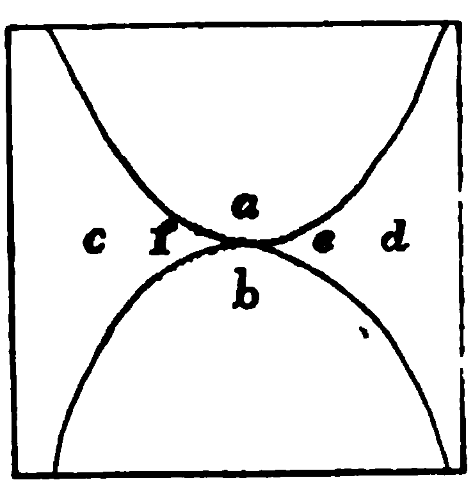

FIG. 1.

This general form of breakage occurs also in non-homogeneous stones when crushed on their beds; but in this case the modification which the grain of the stone produces must be taken into account.

9. Cushions. The nature of the material in contact with the stone while under pressure is a matter of great moment. If the materials which press upon the top and bottom of the specimen are soft and yielding and press out sidewise, they introduce horizontal forces which materially diminish the apparent crushing strength of the stone. If the pressing surfaces are hard and unyielding, the resistance of these surfaces adds considerable to the apparent strength. The thing sought is to secure the same conditions in

the experiments that the stone is under in the wall of masonry. It would at first seem that the pressing surface should be of the same kind of stone as the specimen to be tested. But the surface must not be continuous. One stone is seldom placed directly over another, since, necessarily, for the strength of the structure, we introduce what is called bond. Usually, then, there are one or two joints under each stone, which may or may not be filled with mortar. A wooden cushion, one quarter inch thick, placed above and below the test specimen most nearly represents the conditions actually existing in the wall, because it spreads mainly in one direction, and but very little transversely thereto.

Steel, wood, lead, and leather have been used as pressing surfaces. Under certain limitations, as noted below, the relative crushing strengths of stones with these different pressing surfaces are 100, 89, 65, and 62 respectively.* The following exceptions to the preceding general rule must be noted. With stones of great hardness and toughness combined, steel and wood give approximately equal results; with very soft stones, the power of the stone to resist crushing is overcome before sufficient pressure has been developed to bring the action of the wood fibers on the stone fairly into play, and consequently the steel and wood again give the same results.

Formerly it was the custom to interpose sheets of lead between stones subjected to compression, the object of this use of the lead being to secure a uniform bearing; but the lead, being soft, squeezes out sidewise, thus introducing lateral forces which cause the stone to fail by splitting instead of by crushing.

Tests of the strength of blocks of stone are useful only in comparing different stones, and give no idea of the strength of structures built of such stone (see § 222) or of the crushing strength of stone in large masses in its native bed (see § 273).

Then, since it is not possible to have the stone under the same conditions while being tested that it is in the actual structure, it is best to test the stone under conditions that can be accurately described and readily duplicated. Therefore it is rapidly coming to be the custom to test the stone between metal pressing-surfaces. Under these conditions the strength of the specimen will vary greatly with the degree of smoothness of its bed-surfaces. Hence,

* Report on Building Stones, in Report of Chief of Engineers, U. S. A., 1875, App. II; also bound separately, page 29.

to obtain definite and precise results, these surfaces should be rubbed or ground perfectly smooth; but as this is tedious and expensive, it is quite common to reduce the bed-surfaces to planes by plastering them with a thin coat of plaster of Paris. With the stronger stones, specimens with plastered beds will show less strength than those having rubbed beds, and this difference will vary also with the length of time the plaster is allowed to harden. With a stone having a strength of 5000 to 6000 pounds per square inch, allowing the plaster to attain its maximum strength, this difference varied from 5 to 20 per cent., the mean for ten trials being almost 10 per cent. of the strength of the specimen with rubbed beds.

10. Form of Test Specimen. The crushing strength of stone is generally obtained by compressing cubes of the material. In good masonry, stones of greater height than the breadth of the bed are never used; therefore the experiments should not be made on objects whose height is more than the breadth, unless columnar strength is desired. Experiments show that slabs of width much greater than height do not give a definite and easily recognized point of failure, but that the cube does allow quite sufficient opportunity for the easily recognized and natural angular breakage of stone. Experiments also show that, while slabs give much greater (sometimes 50 per cent. more) resistance than cubes, those higher than wide, to the limit of nearly three times the width, are but little, if any, decreased in resistance by increased altitude. In the Hodgkinson-Fairbairn experiments on cast iron, the conclusion was drawn that the test specimen should have a height of at least 1½ times its diameter. This conclusion is often applied to stone, but is nevertheless inapplicable, since stone is generally employed in such a position that the height is less than the breadth.

11. Size of Cube. Although the cube is the form of test specimen generally adopted, there is not equal unanimity as to size of the cube; hence it is necessary to inquire into the effect of the size of the specimen upon the apparent strength. This inquiry is necessary before we can compare results by different experimenters; and it is important as indicating the relation between the strength of the test specimen and that of the block of stone as laid in the wall.

Hodgkinson tested teak-wood cylinders, ½ inch, 1 inch, and 2 inches in diameter, the height in each case being twice the diameter, and obtained results which seemed to show that the strength per square inch was independent of the bed-area.

On the other hand, Gen. Gillmore tested 1-inch, 2-inch, and 3-inch cubes of Michigan pine, crushed against the ends of the fiber, which gave a considerable increase in strength per square inch of bed-area as the size of the cube increased. Gillmore also made experiments upon cubes of stone varying from $\frac{1}{4}$ inch to 4 inches on a side, with the result that the relation between the crushing strength and the size of the cube can be expressed by the formula

$$y = a\sqrt[3]{x},$$

in which y is the total crushing pressure in pounds per square inch of bed-area, a is the crushing pressure of a 1-inch cube of the same material, and x is the length in inches of an edge of the cube under trial. For two samples of Berea (Ohio) sandstone, a was 7000 and 9500 lbs. respectively.*

According to tests made with the large and accurate testing-machine at the Watertown (Mass.) Arsenal † on cubes of neat cement, cement mortars, concrete, and Haverstraw sandstone, the strength of cubes appears to be independent of the bed-area.‡ Various other experimenters have uniformly obtained like results. There is no sufficient reason for believing that the strength of cubes of homogeneous stones varies with the size of the cube. The experiments which indicated a variation of strength with size were doubtless in error, owing probably to the friction of the machine.

12. Dressing the Specimen. It is well known that even large stones can be broken by striking a number of comparatively light blows along any particular line, in which case the force of the blows gradually weakens the cohesion of the particles. This principle finds application in the preparation of test specimens of stone. If the specimen is dressed by hand, the concussion of the tool greatly affects its internal conditions, particularly with test specimens of small dimensions. With 2-inch cubes, the tool-dressed specimen usually shows only about 60 per cent. of the strength of the sawed

* Report on Strength of Building Stone, Appendix II, Report of Chief of Engineers of U. S. A. for 1875.

† Report on the "Tests of Metals, etc.," for the year ending June 30, 1884, pp. 126, 166, 167, 197, 212, 213, 215; the same being Sen. Ex. Doc. No. 35, 49th Cong., 1st Session.

‡ For a more elaborate discussion of this point by the author, see *Engineering News* vol. xix. pp. 511-12.

sample. The sawed sample most nearly represents the conditions of actual practice.

Unfortunately, experimenters seldom state whether the specimens were tool-dressed or sawed. The disintegrating effect of the tool in dressing is greater with small than with large specimens. This may account in part for the difference in strength of different sizes of test specimens. All stones are strongest when laid on their bed, *i.e.*, in their natural position; and with sedimentary rocks there is a very great difference in the two positions. Hence, in preparing the specimen the natural bed should be marked, and the position in which it is tested should be noted. Tests of the strength of blocks of stone are useful only in comparing different stones, and give no idea of the strength of structures built of such stone (see § 222) or of the crushing strength of stone in large masses in its natural bed (see § 273).

13. Data on Crushing Strength. The strength of the principal classes of building stone in use in the United States is about as follows:

TABLE 1.

CRUSHING STRENGTH OF CUBES OF STONE.

KINDS OF STONE.	ULTIMATE CRUSHING STRENGTH.			
	Pounds per Square Inch.		Tons per Square Foot.	
	Min.	Max.	Min.	Max.
Trap Rocks of N. J.	20,000	24,000	1,440	1,730
Granite	12,000	21,000	860	1,510
Marble	8,000	20,000	580	1,440
Limestone	7,000	20,000	500	1,440
Sandstone	5,000	15,000	360	1,080

14. Crushing Strength of Slabs. Only a few experiments have been made to determine the crushing strength of slabs of stone. The strength per square inch of bed-surface was considerably greater than that for cubes; but a study of the results of all of the reliable experiments * fails to discover any simple relation between

* See Report on "Tests of Metals, etc.," for 1884.—Sen. Ex. Doc. No. 35, 49th Cong., 1st Session,—pp. 126 and 212.

the crushing strength of cubes and slabs. It is probable that the effect of the pressing surface is so great as to completely mask the variation due to height of specimen. More experiments on this subject are very much needed.

15. Transverse Strength. When stones are used for lintels, etc., their transverse strength becomes important. The ability of a stone to resist as a beam depends upon its tensile strength, since that is always much less than its compressive strength. A knowledge of the relative tensile and compressive strength of stones is valuable in interpreting the effect of different pressing surfaces in compressive tests, and also in determining the thickness required for lintels, sidewalks, cover-stones for box culverts, thickness of footing courses, etc.

Owing to the small cross section of the specimen employed in determining the transverse strength of stones,—usually a bar 1 inch square,—the manner of dressing the sample affects the apparent transverse strength to a greater degree than the compressive strength (see § 12); and it is even more unfortunate, since the strength of the stone as used in actual practice is nearly proportional to the strength of sawed samples.

The following formulas are useful in computing the breaking load of a slab of stone. Let W represent the concentrated center load *plus* half of the weight of the beam itself, in pounds; and let b, d, and l represent the breadth, depth, and length, in inches, respectively. Let R = the modulus of rupture, in lbs. per sq. in.; let C = the weight, in pounds, required to break a bar 1 inch square and 1 foot long between bearings; and let L = the length of the beam in feet. Then

$$W = \frac{2\,b\,d^2}{3\,l} R = \frac{b\,d^2}{L} C.$$

The equivalent uniformly distributed weight is equal to twice the concentrated center load.

Table 2 on the following page gives the values of R, the modulus of rupture, and of C, the co-efficient of transverse strength, required in the above formulas.

Example.—To illustrate the method of using the above formulas, assume that it is desired to know the breaking load for a limestone slab 3 inches thick, 4 feet wide, and 6 feet long. Then $b = 48$;

TABLE 2.

TRANSVERSE STRENGTH OF STONE, BRICK, AND MORTAR.

Material.	Modulus of Rupture.			Co-efficient of Transverse Strength.		
	Max.	Min.	Aver.	Max.	Min.	Aver.
Blue-stone flagging.............	4,511	360	2,700	251	20	150
Granite..........................	2,700	900	1,800	150	50	100
Limestone......................	2,500	140	1,500	140	8	83
" oölitic, from Ind., sawed.	2,590	2,190	2,338	144	122	130
Marble..........................	2,880	144	2,160	160	8	120
Sandstone.......................	2,340	576	1,260	130	32	70
Slate............................	9,000	1,800	5,400	500	100	300
Brick (§ 59).....................	1,796	269	800	100	15	45
Concrete—see § 156............						
Mortar, neat Portland, 1 year old..			1,158*			64*
Mortar, 1 part Portland cement, 1 part sand, 1 year old..........			945*			52*
Mortar, 1 part Portland cement, 2 parts sand, 1 year old..........			682*			38*
Mortar, neat Rosendale, 1 year old.	715	415	600	39	23	33
Mortar, 1 part Rosendale cement, 1 part sand, 1 year old..........	690	348	526	38	19	29
Mortar, 1 part Rosendale cement, 2 parts sand, 1 year old.........	479	338	405	26	18	22

$d = 3$; $l = 72$; $R = 1500$ lbs.,—the "average" value from the table;—and $C = 83$. Substituting these values, we have

$$W = \frac{2\,b\,d^2}{3\,l}R = \frac{2 \times 48 \times 9}{3 \times 72}1500 = 6000 \text{ pounds};$$

or, using the other form,

$$W = \frac{b\,d^2}{L}C = \frac{48 \times 9}{6}83 = 5976 \text{ pounds},$$

which agrees with the preceding except for omitted decimals. Hence the breaking load for average quality of limestone is 6000 pounds concentrated along a line half-way between the ends; the uniformly distributed load is twice this, or 12,000 pounds. The

* Only one experiment.

question of what margin should be allowed for safety is one that can not be determined in the abstract; it depends upon the accuracy with which the maximum load is estimated, upon the manner the load is applied—whether with shock or not,—upon the care with which the stone was selected, etc. This subject will be discussed further in connection with the use of the data of the above table in subsequent parts of this volume.

16. ELASTICITY. But very few experiments have been made to determine the co-efficient of elasticity, the elastic limit, and the "set" of stone. Data on these points would be valuable in determining the effect of combining masonry and metal, of joining different kinds of masonry, or of joining new masonry to old; in calculating the effect of loading a masonry arch; in proportioning abutments and piers of railroad bridges subject to shock, etc. The following is all the data that can be found:

TABLE 3.

CO-EFFICIENT OF ELASTICITY OF STONE, BRICK, AND MORTAR.

MATERIAL.	POUNDS PER SQUARE INCH.
Haverstraw Freestone *	950,000
Portland Stone (oölite limestone)†	1,530,000
Marble†	2,500,000
Portland Granite‡	5,500,000
Slate†	7,000,000
Grafton Limestone‡	8,000,000
Richmond Granite‡	13,000,000
Brick, medium—mean of 16 experiments*	3,500,000
Louisville Cement Mortar, 4 months old: ‡	
Neat cement	800,000
1 part cement, 1 part sand	600,000
1 part cement, 2 parts sand	1,300,000
Ulster Co. (N. Y.) Cement Mortar, 22 months old: *	
2 parts cement, 3 parts sand	640,000
1 part cement, 3 parts sand	535,000
Portland Cement Mortar, 22 months old *	1,525,000

* U. S. testing-machine, Watertown, Mass. † Tredgold, as quoted by Stoney.
‡ History of St. Louis Bridge, pp. 324–28.

17. Hardness and Toughness. The reader should notice that these two qualities are essentially different. Hardness of itself is not necessarily an element of durability. Both qualities should exist in a stone to be used for stoops, pavements, road-metal, the facing of piers, etc. No experiments have been made in this country to test the resisting power of stone when exposed to the different kinds of service. A table of the resistance of stones to abrasion is often quoted,* but as it contains only foreign stones, which are described by local names, it is not of much value.

18. Bibliographical. A large number of tests have been applied to the building stones of the United States. For additional results and details, see Gillmore's Report on the Building Stones of the U. S., Appendix II. of the Annual Report of the Chief of Engineers, U. S. A., for 1875; Vol. X. of the Tenth Census of the U. S., Report on the Quarry Industry, pp. 330–35; History of the St. Louis Bridge, Chapter 25; Transactions American Society of Civil Engineers, Vol. II. pp. 145–51; Vol. I. of the Report of the Geological Survey of Minnesota; Report for 1884 of "Tests of Metals, etc.," made at the Watertown Arsenal, under the direction of the Chief of Ordnance, U. S. A.; and the Reports of the various State Geological Surveys, and of the commissioners of the various State capitols and of other public buildings.

From what has been said above, it is clear that the results should be used only with a knowledge of the form and size of the specimen and also a clear understanding of the method of making the experiments.

Durability.

19. "Although the art of building has been practiced from the earliest times, and constant demands have been made in every age for the means of determining the best materials, yet the process of ascertaining the durability of stone appears to have received but little definite scientific attention, and the processes usually employed for solving this question are still in a very unsatisfactory state. Hardly any department of technical science is so much neglected as that which embraces the study of the nature of stone, and all the varied resources of lithology in chemical, microscopical, and physical methods of investigation, wonderfully developed within the last

* For example, Mahan's Civil Engineering, p. 13.

quarter century, have never yet been properly applied to the selection and protection of stone used for building purposes."*

Examples of the rapid decay of building stones have already been referred to, and numerous others could be cited, in which a stone which it was supposed would last forever has already begun to decay. In every way, the question of durability is of more interest to the architect than to the engineer; although it is of enough importance to the latter to warrant a brief discussion here.

20. DESTRUCTIVE AGENTS. The destructive agents may be classified as mechanical, chemical, and organic. The last are unimportant, and will not be considered here.

21. Mechanical Agents. For our climate the mechanical agents are the most efficient. These are frost, wind, rain, fire, pressure, and friction.

The action of frost is usually one of the main causes of rapid decay. Two elements are involved,—the friability of the material and its power of absorbing moisture. In addition to the alternate freezing and thawing, the constant variations of temperature from day to day, and even from hour to hour, give rise to molecular motions which affect the durability of stone as a building material. This effect is greatest in isolated columns,—as monuments, bridge piers, etc.

The effect of rain depends upon the solvent action of the gases which it contains, and upon its mechanical effect in the wear of pattering drops and streams trickling down the face of the wall.

A gentle breeze dries out the moisture of a building stone and tends to preserve it; but a violent wind wears it away by dashing sand grains, street dust, ice particles, etc., against its face. The extreme of such action is illustrated by the vast erosion of the sandstone in the plateaus of Colorado, Arizona, etc., into tabular *mésas*, isolated pillars, and grotesquely-shaped hills, by the erosive force of sand grains borne by the winds. The effect is similar to that of the sand blast as used in various processes of manufacture. A violent wind also forces the rain-water, with all the corrosive acids it contains, into the pores of stones, and carries off the loosened grains, thus keeping a fresh surface of the stone exposed. Again, the swaying of tall edifices by the wind must cause a continual motion,

* Tenth Census of the U. S., Vol. X., Report on the Quarry Industry, p. 364.

not only in the joints between the blocks, but among the grains of the stones themselves. Many of these have a certain degree of flexibility, it is true; and yet the play of the grains must gradually increase, and a tendency to disintegration result.

Experience in great fires in the cities shows that there is no stone which can withstand the fierce heat of a mass of burning buildings. Sandstones seem to be the least affected by great heat, and granite most.

Friction affects sidewalks, pavements, etc., and has already been referred to (§ 17). It would also affect bridge piers, sea-walls, docks, etc.

The effect of pressure in destroying stone is one of the least importance, provided the load to be borne does not too nearly equal the crushing strength. The pressure to which stone is subjected does not generally exceed one tenth of the ultimate strength as determined by methods already described.

22. Chemical Agents. The principal ones are acids. Every constituent of stone, except quartz, is subject to attack by acids; and the carbonates, which enter as chief constituents or as cementing materials, yield very readily to such action. Oxygen and ammonia by their chemical action tend to destroy stones. In cities or manufacturing districts sulphur acids and carbonic acid have a very marked effect. These all result from the combustion of gas, coal, etc., and some are also the residuary gases of many kinds of manufactories. The nitric acid in the rain and the atmosphere exerts a perceptible influence in destroying building stone.

23. Resisting Agents. The durability of a building stone depends upon three conditions; viz., the chemical and mineralogical nature of its constituents, its physical structure, and the character and position of its exposed surfaces.

24. Chemical Composition. The chemical composition of the principal constituent mineral and of the cementing material has an important effect upon the durability of a stone.

A siliceous stone, other things being equal, is more durable than a limestone; but the durability of the former plainly depends upon the state of aggregation of the individual grains and their cementing bond, as well as on the chemical relation of the silica to the other chemical ingredients. A dolomitic limestone is more durable than a pure limestone.

A stone that absorbs moisture abundantly and rapidly is apt to be injured by alternate freezing and thawing; hence clayey constituents are injurious. An argillaceous stone is generally compact, and often has no pores visible to the eye; yet such will disintegrate rapidly either by freezing and thawing, or by corrosive vapors.

The presence of calcium carbonate, as in some forms of marble and in earthy limestones, renders a building material liable to rapid attack by acid vapors. In some sandstones the cementing material is the hydrated form of ferric oxide, which is soluble and easily removed. Sandstones in which the cementing material is siliceous are likely to be the most durable, although they are not so easily worked as the former. A stone that has a high per cent. of alumina (if it be also non-crystalline), or of organic matter, or of protoxide of iron, will usually disintegrate rapidly. Such stones are generally of a bluish color.

25. Seasoning. The thorough drying of a stone before, and the preservation of this dryness after, its insertion in masonry are commonly recognized as important factors of its durability; but the exact nature of the process of seasoning, and of the composition of the quarry-sap removed by thorough drying, have never been determined. The quarry water may contain little else than ordinary well-water, or may be a solution more or less nearly saturated, at the ordinary temperature, with carbonate of calcium, silica, double salts of calcium and magnesium, etc. In the latter case, hardening results from the drying, and an exact knowledge of its nature might throw important light on the best means for the artificial preservation of stone. Again, water may exist in large quantity, in chemical combination, in the silicates (e.g., chlorite, kaolin, etc.), or in the hydrated iron oxides which constitute the cement of a building stone.

26. Physical Structure. The physical properties which contribute to durability are hardness, toughness, homogeneity, contiguity of the grains, and the structure—whether crystalline or amorphous.

Although hardness (resistance to crushing) is often regarded as the most important element, yet resistance to weathering does not necessarily depend upon hardness alone, but upon hardness and the non-absorbent properties of the stone. A hard material of close and firm texture is, however, in those qualities at least, especially

fitted to resist friction, as in stoops, pavements, and road metal, and the wear of rain-drops, dripping rain-water, the blows of the waves, etc.

Porosity is an objectionable element. An excessive porosity increases the layer of decomposition which is caused by the acids of the atmosphere and of the rain, and also deepens the penetration of frost and promotes its work of disintegration.

If the constituents of a rock differ greatly in hardness, texture, solubility, porosity, etc., the weathering is unequal, the surface is roughened, and the sensibility of the stone to the action of frost is increased.

The principle which obtains in applying an artificial cement, such as glue, in the thinnest film in order to secure the greatest binding force finds its analogy in the building stones. The thinner the films of the natural cement and the closer the grains of the predominant minerals, the stronger and more durable the stone. One source of weakness in the famous brown-stone of New York City lies in the separation of the rounded grains of quartz and feldspar by a superabundance of ocherous cement. Of course the further separation produced by fissure, looseness of lamination, empty cavities and geodes, and excess of mica tends to deteriorate still further a weak building stone.

Experience has generally shown that a crystalline structure resists atmospheric attack better than an amorphous one. This principle has been abundantly illustrated in the buildings of New York City. The same fact is generally true with the sedimentary rocks also, a crystalline limestone or good marble resisting erosion better than earthy limestone. A stone that is compactly and finely granular will exfoliate more easily by freezing and thawing than one that is coarse-grained. A stone that is laminar in structure absorbs moisture unequally and will be seriously affected by unequal expansion and contraction,—especially by freezing and thawing. Such a stone will gradually separate into sheets. A stone that has a granular texture, as contrasted with one that is crystalline or fibrous, will crumble sooner by frost and by chemical agents, because of the easy dislodgment of the individual grains.

The condition of the surface, whether rough or polished, influences the durability,—the smoother surface being the better.

The stone is more durable if the exposed surface is vertical than if inclined. The lamination of the stone should be horizontal.

27. Methods of Testing Durability. It has long been recognized that there are two ways in which we can form a judgment of the durability of a building stone, and these may be distinguished as natural and artificial.

28. Natural Methods. These must always take the precedence whenever they can be used, because they involve (1) the exact agencies concerned in the atmospheric attack upon stone, and (2) long periods of time far beyond the reach of artificial experiment.

One method is to visit the quarry and observe whether the ledges that have been exposed to the weather are deeply corroded, or whether these old surfaces are still fresh. In applying this test, consideration must be given to the modifying effect of geological phenomena. It has been pointed out that "the length of time the ledges have been exposed, and the changes of actions to which they may have been subjected during long geological periods, are unknown; and since different quarries may not have been exposed to the same action, they do not always afford definite data for reliable comparative estimates of durability, except where different specimens occur in the same quarry."

North of the glacial limit, all the products of decomposition have been planed away and deposited as drift-formation over the length and breadth of the land. The rocks are therefore, in general, quite fresh in appearance, and possess only a slight depth of cap or worthless rock. The same classes of rock, however, in the South are covered with rotten products from long ages of atmospheric action.

A study of the surfaces of old buildings, bridge piers, monuments, tombstones, etc., which have been exposed to atmospheric influences for years, is one of the best sources of reliable information concerning the durability of stone. A durable stone will retain the tool-marks made in working it, and preserve its edges and corners sharp and true.

29. Artificial Methods of Testing Durability. The older but less satisfactory methods are: determining (1) the resistance to crushing, (2) the absorptive power, (3) the resistance to the expansion of frost, by saturating the stone with some solution which will crystallize in the pores of the stone and produce an effect similar to frost, (4) the solubility in acids, and (5) microscopical examination.

30. Absorptive Power. The ratio of absorption depends largely on the density,—a dense stone absorbing less water than a lighter, more porous one. Compactness is therefore a matter of importance, especially in cold climates; for if the water in a stone is once allowed to freeze, it destroys the surface, and the stone very speedily crumbles away. Other things being equal, the less the absorption the better the stone.

To determine the absorptive power, dry the specimen and weigh it carefully; then soak it in water for 24 hours, and weigh again. The increase in weight will be the amount of absorption. Table 4 shows the weight of water absorbed by the stone as compared with the weight of the dry stone—that is, if 300 units of dry stone weigh 301 units after immersion, the absorption is 1 in 300, and is recorded as 1–300.

Dr. Hiram A. Cutting, State Geologist of Vermont, determined the absorptive power * by placing the specimens in water under the receiver of an air-pump, and found the ratio of absorption a little larger than is given in the following table. It is believed that the results given below more nearly represent the conditions of actual practice. The values in the "Max." column are the means of two or three of the largest results, and those in the "Min." column of two or three of the smallest. The value in the last column is the mean for 20 or more specimens.

TABLE 4.

ABSORPTIVE POWER OF STONE, BRICK, AND MORTAR.

KIND OF MATERIAL.	RATIO OF ABSORPTION.		
	Max.	Min.	Average.
Granites	1–150	0	1–750
Marbles	1–150	0	1–300
Limestones	1–20	1–500	1–38
Sandstones	1–15	1–240	1–24
Bricks	1–4	1–50	1–10
Mortars	1–2	1–10	1–4

31. Effect of Frost. To determine the probable effect of frost upon a stone, carefully wash, dry, and weigh samples, and then wet

* Van Nostrand's Engin'g Mag., vol. xxiv. pp. 491–95.

them and expose to alternate freezing and thawing, after which wash, dry, and weigh again. The loss in weight measures the relative durability.

A quicker way of accomplishing essentially the same result is to heat the specimens to 500° or 600° F., and plunge them, while hot, into cold water. The following comparative results were obtained by the latter method :*

	Relative Ratio of Loss.
White brick	1
Red brick	2
Brown-stone (sandstone from Conn.)	5
Nova Scotia sandstone	14

32. Brard's Test. Brard's method of determining the effect of frost is much used, although it does not exactly conform to the conditions met with in nature. It consists in weighing carefully some small pieces of the stone, which are then boiled in a concentrated solution of sulphate of soda and afterwards hung up for a few days in the open air. The salt crystallizes in the pores of the stone, expands, and produces an effect somewhat similar to frost, as it causes small pieces to separate in the form of dust. The specimens are again weighed, and those which suffer the smallest loss of weight are the best. The test is often repeated several times. It will be seen that this method depends upon the assumption that the action of the salt in crystallizing is similar to that of water in freezing. This is not entirely correct, since it substitutes chemical and mechanical action for merely mechanical, to disintegrate the stone, thus giving the specimen a worse character than it really deserves. The following results were obtained by this method: †

	Relative Ratio of Loss.
Hard brick	1
Light dove-colored sandstone from Seneca, Ohio	2
Coarse-grained sandstone from Nova Scotia	2
Coarse-grained sandstone from Little Falls, N. J.	5
Coarse dolomite marble from Pleasantville, N. Y.	7
Coarse-grained sandstone from Conn	13
Soft brick	16
Fine-grained sandstone from Conn	19

* Tenth Census of the U. S., vol. x., Report on the Quarry Industry, p. 384. For a table showing essentially the same results, see Van Nostrand's Engin'g Mag., vol. xiv. p. 537.

† Tenth Census, vol. x., Report of the Quarry Industry, p. 385.

33. Effect of the Atmosphere. To determine the effect of the atmosphere of a large city, where coal is used for fuel, soak clean small pieces of the stone for several days in water which contains one per cent. of sulphuric and hydrochloric acids, agitating frequently. If the stone contains any earthy matter likely to be dissolved by the gases of the atmosphere, the water will be more or less cloudy or muddy. The following results were obtained by this method: *

	Relative Ratio of Loss.
White brick	1
Red brick	5
Nova Scotia stone	9
Brown-stone	80

34. Microscopical Examination. It is now held that the best method of determining the probable durability of a building stone is to study its surface, or thin, transparent slices, under a microscope. This method of study in recent years has been most fruitful in developing interesting and valuable knowledge of a scientific and truly practical character. An examination of a section by means of the microscope will show, not merely the various substances which compose it, but also the method according to which they are arranged and by which they are attached to one another. For example, "pyrites is considered to be the enemy of the quarryman and constructor, since it decomposes with ease, and stains and discolors the rock. Pyrites in sharp, well-defined crystals sometimes decomposes with great difficulty. If a crystal or grain of pyrites is embedded in soft, porous, light-colored sandstones, like those which come from Ohio, its presence will with certainty soon demonstrate itself by the black spot which will form about it in the porous stone, and which will permanently disfigure and mar its beauty. If the same grain of pyrites is situated in a very hard, compact, non-absorbent stone, the constituent minerals of which are not rifted or cracked, this grain of pyrites may decompose and the products be washed away, leaving the stone untarnished."

35. Methods of Preserving. Vitruvius, the Roman architect, two thousand years ago recommended that stone should be quarried in summer when driest, and that it should be seasoned by being allowed to lie two years before being used, so as to allow the natural

* Tenth Census, vol. x., Report on the Quarry Industry, p. 385.

sap to evaporate. It is a notable fact, that in the erection of St. Paul's Cathedral in London, England, Sir Christopher Wren required that the stone, after being quarried, should be exposed for three years on the sea-beach, before its introduction into the building.

The surfaces of buildings are often covered with a coating of paint, coal-tar, oil, paraffine, soap and alum, rosin, etc., to preserve them.

Another method of treatment consists in bathing the stone in successive solutions, the chemical actions bringing about the formation of insoluble silicates in the pores of the stone. For example, if a stone front is first washed with an alkaline fluid to remove dirt, and this followed by a succession of baths of silicate of soda or potash, and the surface is then bathed in a solution of chloride of lime, an insoluble lime silicate is formed. The soluble salt is then washed away, and the insoluble silicate forms a durable cement and checks disintegration. If lime-water is substituted for chlorate of lime, there is no soluble chlorate to wash away.

There are a great many applications that have been used for the prevention of the decay of building stones, as paint, oil, coal-tar, bees-wax, rosin, paraffine, etc., and numerous chemical preparations similar to that mentioned in the paragraph just above; but all are expensive, and none have proved fairly satisfactory.*

It has already been stated that, in order to resist the effects of both pressure and weathering, a stone should be placed on its natural bed. This simple precaution adds considerably to the durability of any stone.

ART. 3. CLASSIFICATION AND DESCRIPTION OF BUILDING STONES.

36. CLASSIFICATION. Building stones are variously classified according to geological position, physical structure, and chemical composition.

37. Geological Classification. The geological position of rocks has but little connection with their properties as building materials. As a general rule, the more ancient rocks are the stronger and the

* For an elaborate and valuable article by Prof. Eggleston on the causes of decay and the methods of preserving building stones, see Trans. Am. Soc. of C. E., vol. xv. pp. 647-704; and for a discussion on the same, see same volume, pp. 705-16.

more durable; but to this there are many exceptions. According to the usual geological classification, rocks are divided into igneous, metamorphic, and sedimentary. Greenstone, basalt, and lava are examples of igneous rocks; granite, marble, and slate, of metamorphic; and sandstone, limestone, and clay, of sedimentary. Although clay can hardly be classed with building stones, it is not entirely out of place in this connection, since it is employed in making bricks and cement, which are important elements of masonry.

38. Physical Classification. With respect to the structural character of large masses, rocks are divided into stratified and unstratified.

In their more minute structure the *unstratified* rocks present, for the most part, an aggregate of crystalline grains, firmly adhering together. Granite, trap, basalt, and lava are examples of this class.

In the more minute structure of *stratified* rocks, the following varieties are distinguished: 1. *Compact crystalline* structure; accompanied by great strength and durability, as in quartz-rock and marble. 2. *Slaty* structure, easily split into thin layers; accompanied by both extremes of strength and durability, clay-slate and hornblende-slate being the strongest and most durable. 3. The *granular crystalline* structure, in which crystalline grains either adhere firmly together, as in gneiss, or are cemented into one mass by some other material, as in sandstone; accompanied by all degrees of compactness, porosity, strength, and durability, the lowest extreme being sand. 4. The *compact granular* structure, in which the grains are too small to be visible to the unaided eye, as in blue limestone; accompanied by considerable strength and durability. 5. *Porous, granular* structure, in which the grains are not crystalline, and are often, if not always, minute shells cemented together; accompanied by a low degree of strength and durability. 6. The *conglomerate* structure, where fragments of one material are embedded in a mass of another, as graywacke; accompanied by all degrees of strength and durability.

A study of the fractured surface of a stone is one means of determining its structural character. The even fracture, when the surfaces of division are planes in definite positions, is characteristic of a crystalline structure. The uneven fracture, when the broken surface presents sharp projections, is characteristic of a granular

structure. The slaty fracture gives an even surface for planes of division parallel to the lamination, and uneven for other directions of division. The conchoidal fracture presents smooth concave and convex surfaces, and is characteristic of a hard and compact structure. The earthy fracture leaves a rough, dull surface, and indicates softness and brittleness.

39. Chemical Classification. Stones are divided into three classes with respect to their chemical composition, each distinguished by the earth which forms its chief constituent; viz., siliceous stones, argillaceous stones, and calcareous stones.

Siliceous Stones are those in which silica is the characteristic earthy constituent. With a few exceptions their structure is crystalline-granular, and the crystalline grains contained in them are hard and durable; hence weakness and decay in them generally arise from the decomposition or disintegration of some softer and more perishable material, by which the grains are cemented together, or, when they are porous, by the freezing of water in their pores. The principal siliceous stones are granite, syenite, gneiss, mica-slate, greenstone, basalt, trap, talc, soapstone, quartz-rock, hornblende-slate, and sandstone.

Argillaceous or Clayey Stones are those in which alumina, although it may not always be the most abundant constituent, exists in sufficient quantity to give the stone its characteristic properties. The principal kinds are porphyry, clay, slate, and graywacke-slate.

Calcareous Stones are those in which carbonate of lime predominates. They effervesce with the dilute mineral acids, which combine with the lime and set free carbonic-acid gas. Sulphuric acid forms an insoluble compound with the lime. Nitric and muriatic acids form compounds with it, which are soluble in water. By the action of intense heat the carbonic acid is expelled in gaseous form, and the lime is left in its caustic or alkaline state, when it is called quicklime. Some calcareous stones consist of pure carbonate of lime; in others it is mixed with sand, clay, and oxide of iron, or combined with carbonate of magnesia. The durability of calcareous stones depends upon their compactness, those which are porous being disintegrated by the freezing of water, and by the chemical action of an acid atmosphere. They are, for the most part, easily wrought. The principal calcareous stones are marble,

compact limestone, granular limestone (the calcareous stone of the geological classification), and magnesian limestone or dolomite.

40. Description of Building Stones. A few of the more prominent classes of building stones will now be briefly described.

41. Trap. Although trap is the strongest of building materials, and exceedingly durable, it is little used, owing to the great difficulty with which it is quarried and wrought. It is an exceedingly tough rock, and, being generally without cleavage or bedding, is especially intractable under the hammer or chisel. It is, however, sometimes used with excellent effect in cyclopean architecture, the blocks of various shapes and sizes being fitted together with no effort to form regular courses. The "Palisades" (the bluff skirting the western shore of the Hudson River, opposite and above New York) are composed of trap-rock,—much used for road-metal, street pavements, and railroad ballast.

42. Granite. Granite is the strongest and most durable of all the stones in common use. It generally breaks with regularity, and may be quarried in simple shapes with facility; but it is extremely hard and tough, and therefore can only be wrought into elaborate forms with a great expenditure of labor. For this reason the use of granite is somewhat limited. Its strength and durability commend it, however, for foundations, docks, piers, etc., and for massive buildings; and for these purposes it is in use the world over.

The larger portion of our granites are some shade of gray in color, though pink and red varieties are not uncommon, and black varieties occasionally occur. They vary in texture from very fine and homogeneous to coarsely porphyritic rocks, in which the individual grains are an inch or more in length. Excellent granites are found in New England, throughout the Alleghany belt, in the Rocky Mountains, and in the Sierra Nevada. Very large granite quarries exist at Vinalhaven, Maine; Gloucester and Quincy, Massachusetts; and at Concord, New Hampshire. These quarries furnish nearly all the granite used in this country. An excellent granite, which is largely used at Chicago and in the Northwest, is found at St. Cloud, Minnesota.

At the Vinalhaven quarry a single block 300 feet long, 20 feet wide, and 6 to 10 feet thick was blasted out, being afterwards broken up. Until recently the largest single block ever quarried and

dressed in this country was that used for the General Wool Monument, now in Troy, New York, which measured, when completed, 60 feet in height by 5½ feet square at the base, being only 9 feet shorter than the Egyptian Obelisk now in Central Park, New York. In 1887 the Bodwell Granite Company took out from its quarries in Maine a granite shaft 115 feet long, 10 feet square at the base, and weighing 850 tons. It is claimed that this is the largest single quarried stone on record.

43. Marbles. In common language, any limestone which will take a good polish is called a marble; but the name is properly applied only to limestones which have been exposed to metamorphic action, and have thereby been rendered more crystalline in texture, and have had their color more or less modified or totally removed. Marbles exhibit great diversity of color and texture. They are pure white, mottled white, gray, blue, black, red, yellow, or mottled with various mixtures of these colors. Marble is confessedly the most beautiful of all building materials, but is chiefly employed for interior decorations.

44. Limestones. Limestones are composed chiefly or largely of carbonate of lime. There are many varieties of limestone, which differ in color, composition, and value for engineering and building purposes, owing to the differences in the character of the deposits and chemical combinations entering into them. "If the rock is compact, fine-grained, and has been deposited by chemical agencies, we have a variety of limestone known as travertine. If it contains much sand, and has a more or less conchoidal fracture, we have a siliceous limestone. If the silica is very fine-grained, it is hornstone. If the silica is distributed in nodules or flakes, either in seams or throughout the mass, it is cherty limestone; if it contains silica and clay in about equal proportions, hydraulic limestone; if clay alone is the principal impurity, argillaceous limestone; if iron is the principal impurity, ferruginous limestone; if iron and clay exceed the lime, ironstone. If the ironstone is decomposed, and the iron hydrated, it is rottenstone; if carbonate of magnesia forms one third or less, magnesian limestone; if carbonate of magnesia forms more than one third, dolomitic limestone."

The lighter-colored and fine-grained limestones, when sawed and used as ashlars, are deservedly esteemed as among our best building materials. They are, however, less easily and accurately worked

under the chisel than sandstones, and for this reason and their greater rarity are far less generally used. The gray limestones, like that of Lockport, New York, when hammer-dressed, have the appearance of light granite, and, since they are easily wrought, they are advantageously used for trimmings in buildings of brick.

Some of the softer limestones possess qualities which specially commend them for building materials. For example, the cream-colored limestone of the Paris basin (*calcaire grossier*) is so soft that it may be dressed with great facility, and yet hardens on exposure, and is a durable stone. Walls laid up of this material are frequently planed down to a common surface, and elaborately ornamented at small expense. The Topeka stone, found and now largely used in Kansas, has the same qualities. It may be sawed out in blocks almost as easily as wood, and yet is handsome and durable when placed in position. The Bermuda stone and *coquina* are treated in the same way.

Large quantities of limestones and dolomites are quarried in nearly all of the Western States. These are mostly of a dull grayish color, and their uses are chiefly local. The light-colored oölitic limestone of Bedford, Indiana, is, however, an exception to this rule. Not only are the lasting qualities fair and the color pleasing, but its fine even grain and softness render it admirably adapted for carved work. It has been very widely used within the last few years. This stone is often found in layers 20 and 30 feet thick, and is much used for bridge piers and other massive work. There are noted limestone quarries at Dayton and Sandusky, Ohio; at Bedford, Ellettsville, and Salem, Indiana; at Joliet, Lemont, Grafton, and Chester, Illinois; and at Cottonwood, Kansas.

45. Sandstones. "Sandstones vary much in color and fitness for architectural purposes, but they include some of the most beautiful, durable, and highly valued materials used in construction. Whatever their differences, they have this in common, that they are chiefly composed of sand—that is, grains of quartz—to a greater or less degree cemented and consolidated. They also frequently contain other ingredients, as lime, iron, alumina, manganese, etc., by which the color and texture are modified. Where a sandstone is composed exclusively of grains of quartz, without foreign matter, it may be snow-white in color. Examples of this variety are known in many localities. They are rarely used for building, though capa-

ble of being employed for that purpose with excellent effect. They have been more generally valued as furnishing material for the manufacture of glass. The color of sandstones is frequently bright and handsome, and constitutes one of the many qualities which have rendered them so popular. It is usually caused by iron; when gray, blue, or green, by the protoxide, as carbonate or silicate; when brown, by the hydrated oxide; when red, by the anhydrous oxide. The purple sandstones usually derive this shade of color from a small quantity of manganese.

"The texture of sandstones varies with the coarseness of the sand of which they are composed, and the degree to which it is consolidated. Usually the material which unites the grains of sand is silica; and this is the best of all cements. This silica has been deposited from solution, and sometimes fills all the interstices between the grains. If the process of consolidation has been carried far enough, or the quartz grains have been cemented by fusion, the sandstone is converted into quartzite,—one of the strongest and most durable of rocks, but, in the ratio of its compactness, difficult to work. Lime and iron often act as cements in sandstones, but both are more soluble and less strong than silica. Hence the finest and most indestructible sandstones are such as consist exclusively of grains of quartz united by siliceous cement. In some sandstones part of the grains are fragments of feldspar, and these, being liable to decomposition, are elements of weakness in the stone. The very fine-grained sandstones often contain a large amount of clay, and thus, though very handsome, are generally less strong than those which are more purely siliceous.

"The durability of sandstones varies with both their physical and chemical composition. When nearly pure silica and well cemented, sandstones are as resistant to weather as granite, and very much less affected by the action of fire. Taken as a whole, they may be regarded as among the most durable of building materials. When first taken from the quarry, and saturated with quarry water (a weak solution of silica), they are frequently very soft, but on exposure become much harder by the precipitation of the soluble silica contained in them.

46. "Since they form an important part of all the groups of sedimentary rocks, sandstones are abundant in nearly all countries; and as they are quarried with great ease, and are wrought with the

hammer and chisel with much greater facility than limestones, granites, and most other kinds of rocks, these qualities, joined to their various and pleasant colors and their durability, have made them the most popular and useful of building stones. In the United States we have a very large number of sandstones which are extensively used for building purposes.

"Among these may be mentioned the *Dorchester stone* of New Brunswick, and *Brown-stone* of Connecticut and New Jersey. These have been much used in the buildings of the Atlantic cities. The latter has been very popular, but experience has shown it to be seriously lacking in durability.

"Among the sandstones most frequently employed in the building of the interior are :—

1. "*The Ohio stone,* derived from the Berea grit, a member of the Lower Carboniferous series in Northern Ohio. The principal quarries are located at Amherst and Berea. The stone from Amherst is generally light drab in color, very homogeneous in texture, and composed of nearly pure silica. It is very resistant to fire and weathering, and is, on the whole, one of the best and handsomest building stones known. The Berea stone is lighter in color than the Amherst, but sometimes contains sulphide of iron, and is then liable to stain and decompose.

2. "*The Waverly sandstone,* also derived from the Lower Carboniferous series, comes from Southern Ohio. This is a fine-grained homogeneous stone of a light-drab or dove color, works with facility, and is very handsome and durable. It forms the material of which many of the finest buildings of Cincinnati are constructed, and is, justly, highly esteemed there and elsewhere.

3. "*The Lake Superior sandstone* is a dark, purplish-brown stone of the Potsdam age, quarried at Bass Island, Marquette, etc. This is rather a coarse stone, of medium strength, but homogeneous and durable, and one much used in the Lake cities.

4. "*The St. Genevieve stone* is a fine-grained sandstone of a delicate drab or straw color, very homogeneous in tone and texture. It is quarried at St. Genevieve, Missouri, and is one of the handsomest of all our sandstones.

5. "*The Medina sandstone,* which forms the base of the Upper Silurian series in Western New York, furnishes a remarkably strong

and durable stone, much used for pavement and curbing in the Lake cities.

6. "*The coal-measures* of Pennsylvania, Ohio, and other Western States supply excellent sandstones for building purposes at a large number of localities. These vary in color from white to dark red or purple, though generally gray or drab. While strong and durable, they are mostly coarser and less handsome than the sandstones which have been enumerated above. This is the source from which are derived the sandstones used in purely engineering structures."*

47. Other Names. There is a great variety of names of more or less local application, derived from the appearance of the stone, the use to which it is put, etc., which it would be impossible to classify. The same stone often passes under entirely different names in different localities; and stones entirely different in their essential characteristics often pass under the same name.

48. Location of Quarries. For further information concerning location of stone quarries, character of product, etc., see Vol. X., Tenth Census, Report on Quarry Industry, pp. 50–101, and also the reports of the various State geological surveys.

Weight of Stone.

49. The following table contains the weight of the stones most frequently met with.

TABLE 5.

Weight of Building Stones.

Kind of Stone.	Pounds per Cubic Foot.		
	Min.	Max.	Mean.
Granites	161	178	167
Limestones	146	174	158
Marbles	157	180	170
Sandstones	127	151	139
Slates	160	175	174

If it is desired to find the exact weight per cubic foot of a given stone, it is generally easier to find its specific gravity first, and then

* Prof. J. S. Newberry.

multiply by 62.4,—the weight, in pounds, of a cubic foot of water. This method obviates, on the one hand, the expense of dressing a sample to regular dimensions, or, on the other hand, the inaccuracy of measuring a rough, irregular piece. Notice, however, that this method determines the weight of a cubic foot of the solid stone, which will be more than the weight of a cubic foot of the material as used for structural purposes. In finding the specific gravity there is some difficulty in getting the correct displacement of porous stones,—and all stones are more or less porous. There are various methods of overcoming this difficulty, which give slightly different results. The following method, recommended by General Gillmore, is most frequently used:

All loose grains and sharp corners having been removed from the sample and its weight taken, it is immersed in water and weighed there after all bubbling has ceased. It is then taken out of the water, and, after being compressed lightly in bibulous paper to absorb the water on its surface, is weighed again. The specific gravity is found by dividing the weight of the dry stone by the difference between the weight of the saturated stone in air and in water. Or expressing this in a formula,

$$\text{Specific gravity} = \frac{W}{W' - W''},$$

in which W represents the weight of dry stone in air, W' the weight of saturated stone in air, W'' the weight of stone immersed in water.

50. Cost of Stone. See §§ 226–38.

CHAPTER II.

BRICK.

51. Brick is made by submitting clay, which has been prepared properly and moulded into shape, to a temperature which converts it into a semi-vitrified mass.

Common brick is a most valuable substitute for stone. Its comparative cheapness, the ease with which it is transported and handled, and the facility with which it is worked into structures of any desired form, are its valuable characteristics. It is, when properly made, nearly as strong as the best building stone. It is but slightly affected by change of temperature or of humidity; and is also lighter than stone.

Notwithstanding the good qualities which recommend brick as a substitute for stone, it is very little used in engineering structures. It is employed in the construction of sewers and bridge piers, and for the lining of tunnels. Brick could many times be profitably substituted for iron, stone, or timber in engineering structures. This is especially true since recent improvements in the process of manufacture have decreased the cost while they have increased the quality and the uniformity of the product. The advantages of employing brick-work instead of stone masonry will be discussed in connection with brick masonry in Chapter VIII. Probably one thing which has prevented the more general use of brick in engineering is the variable quality of the product and the trouble of proper inspection.

52. Process of Manufacture. The Clay. The quality of the brick depends primarily upon the kind of clay. Common clays, of which the common brick is made, consist principally of silicate of alumina; but they also usually contain lime, magnesia, and oxide of iron. The latter ingredient is useful, improving the product by giving it hardness and strength; hence the red brick of the Eastern States is often of better quality than the white and yellow brick made in the West. Silicate of lime renders the clay too fusible,

and causes the bricks to soften and to become distorted in the process of burning. Carbonate of lime is certain to decompose in burning, and the caustic lime left behind absorbs moisture, prevents the adherence of the mortar, and promotes disintegration.

Uncombined silica, if not in excess, is beneficial, as it preserves the form of the brick at high temperatures. In excess it destroys the cohesion, and renders the bricks brittle and weak. Twenty-five per cent. of silica is a good proportion.

53. Moulding. In the old process the clay is tempered with water and mixed to a plastic state in a pit with a tempering wheel, or in a primitive pug-mill; and then the soft, plastic clay is pressed into the moulds by hand. This method is so slow and laborious that it has been almost entirely displaced by more economical and expeditious ones in which the work is done wholly by machinery. There is a great variety of machines for preparing and moulding the clay, which, however, may be grouped into three classes, according to the condition of the clay when moulded: (1) soft-mud machines, for which the clay is reduced to a soft mud by adding about one quarter of its volume of water; (2) stiff-mud machines, for which the clay is reduced to a stiff mud; and (3) dry-clay machines, with which the dry, or nearly dry, clay is forced into the moulds by a heavy pressure without having been reduced to a plastic mass. These machines may also be divided into two classes, according to the method of filling the moulds: (1) Those in which a continuous stream of clay is forced from the pug-mill through a die and is afterwards cut up into bricks; and (2) those in which the clay is forced into moulds moving under the nozzle of the pug-mill.

54. Burning. The time of burning varies with the character of the clay, the form and size of kiln, and the kind of fuel. With the older processes of burning, the brick, when dry enough, is built up in sections—by brick-makers called "arches,"—which are usually about 5 bricks (3½ feet) wide, 30 to 40 bricks (20 to 30 feet) deep, and from 35 to 50 courses high. Each section or "arch" has an opening—called an "eye"—at the bottom in the center of its width, which runs entirely through the kiln, and in which the fuel used in burning is placed. After the bricks are thus stacked up, the entire pile is enclosed with a wall of green brick, and the joints between the casing bricks are carefully stopped with mud. Burning, including drying, occupies from 6 to 15 days. The brick is first subjected

to a moderate heat, and when all moisture has been expelled, the heat is increased slowly until the "arch-brick," *i. e.*, those next to the "eye," attain a white heat. This temperature is kept up until the burning is complete. Finally, all openings are closed, and the mass slowly cools.

With the more modern processes of burning, the principal yards have permanent kilns. These are usually either a rectangular space surrounded, except for very wide doors at the ends, by permanent brick walls having fire-boxes on the outside; or the kiln may be entirely enclosed—above as well as on the sides—with brick masonry. The latter are usually circular, and are sometimes made in compartments, each of which has a separate entrance and independent connection with the chimney. The latter may be built within the kiln or entirely outside, but a downward draught is invariably secured. The fuel, usually fine coal, is placed near the top of the kiln, and the down draught causes a free circulation of the flame and heated gases about the material being burned. While some compartments are being fired others are being filled, and still others are being emptied.

55. Fire Brick. Fire bricks are used whenever very high temperatures are to be resisted. They are made either of a very nearly pure clay, or of a mixture of pure clay and clean sand, or, in rare cases, of nearly pure silica cemented with a small proportion of clay. The presence of oxide of iron is very injurious, and, as a rule, the presence of 6 per cent. justifies the rejection of the brick. In specifications it should generally be stipulated that fire brick should contain less than 6 per cent. of oxide of iron, and less than an aggregate of 3 per cent. of combined lime, soda, potash, and magnesia. The sulphide of iron—pyrites—is even worse in its effect on fire brick than the substances first named.

When intended to resist only extremely high heat, silica should be in excess; and if to be exposed to the action of metallic oxides, which would tend to unite with silica, alumina should be in excess.

Good fire brick should be uniform in size, regular in shape, homogeneous in texture and composition, easily cut, strong, and infusible.

56. Classification of Common Brick. Bricks are classified according to (1) the way in which they are moulded; (2) their position in the kiln while being burned; and (3) their form or use.

1. The method of moulding gives rise to the following terms:

Soft-mud Brick. One moulded from clay which has been reduced to a soft mud by adding water. It may be either hand-moulded or machine-moulded.

Stiff-mud Brick. One moulded from clay in the condition of stiff mud. It is always machine-moulded.

Pressed Brick. One moulded from dry or semi-dry clay.

Re-pressed Brick. A soft-mud brick which, after being partially dried, has been subjected to an enormous pressure. It is also called, but less appropriately, pressed brick. The object of the re-pressing is to render the form more regular and to increase the strength and density.

Slop Brick. In moulding brick by hand, the moulds are sometimes dipped into water just before being filled with clay, to prevent the mud from sticking to them. Brick moulded by this process is known as slop brick. It is deficient in color, and has a comparatively smooth surface, with rounded edges and corners. This kind of brick is now seldom made.

Sanded Brick. Ordinarily, in making soft-mud brick, sand is sprinkled into the moulds to prevent the clay from sticking; the brick is then called sanded brick. The sand *on the surface* is of no serious advantage or disadvantage. In hand-moulding, when sand is used for this purpose, it is certain to become mixed with the clay and occur in streaks in the finished brick, which is very undesirable; and owing to details of the process, which it is here unnecessary to explain, every third brick is especially bad.

Machine-made Brick. Brick is frequently described as "machine-made;" but this is very indefinite, since all grades and kinds are made by machinery.

2. When brick was generally burned in the old-style up-draught kiln, the classification according to position was important; but with the new styles of kilns and improved methods of burning, the quality is so nearly uniform throughout the kiln, that the classification is less important. Three grades of brick are taken from the old-style kiln:

Arch or Clinker Bricks. Those which form the tops and sides of the arches in which the fire is built. Being over-burned and partially vitrified, they are hard, brittle, and weak.

Body, Cherry, or Hard Bricks. Those taken from the interior of the pile. The best bricks in the kiln.

Salmon, Pale, or Soft Bricks. Those which form the exterior of the mass. Being underburned, they are too soft for ordinary work, unless it be for filling. The terms *salmon* and *pale* refer to the color of the brick, and hence are not applicable to a brick made of a clay that does not burn red. Although nearly all brick clays burn red, yet the localities where the contrary is true are sufficiently numerous to make it desirable to use a different term in designating the *quality*. There is, necessarily, no relation between color, and strength and density. Brick-makers naturally have a prejudice against the term *soft brick,* which doubtless explains the nearly universal prevalence of the less appropriate term—salmon.

3. The form or use of bricks gives rise to the following classification:—

Compass Brick. Those having one edge shorter than the other. Used in lining shafts, etc.

Feather-edge Brick. Those of which one edge is thinner than the other. Used in arches; and more properly, but less frequently, called *voussoir brick.*

Face Brick. Those which, owing to uniformity of size and color, are suitable for the face of the wall of buildings. Sometimes face bricks are simply the best ordinary brick; but generally the term is applied only to re-pressed or pressed brick made specially for this purpose. They are a little larger than ordinary bricks (§ 62).

Sewer Brick. Ordinary hard brick, smooth, and regular in form.

Paving Brick. Very hard, ordinary brick. A vitrified clay block, very much larger than ordinary brick, is sometimes used for paving, and is called a paving brick, but more often, and more properly, a *brick paving-block.*

57. **Requisites for Good Brick.** 1. A good brick should have plane faces, parallel sides, and sharp edges and angles. 2. It should be of fine, compact, uniform texture; should be quite hard; and should give a clear ringing sound when struck a sharp blow. 3. It should not absorb more than one tenth of its weight of water. 4. Its specific gravity should be 2 or more. 5. The crushing strength of half brick, when ground flat and pressed between thick metal

plates, should be at least 7,000 pounds per square inch. 6. Its modulus of rupture should be at least 1,000 pounds per square inch.

1. In regularity of form re-pressed brick ranks first, dry-clay brick next, then stiff-mud brick, and soft-mud brick last. Regularity of form depends largely upon the method of burning.

2. The compactness and uniformity of texture, which greatly influence the durability of brick, depend mainly upon the method of moulding. As a general rule, hand-moulded bricks are best in this respect, since the clay in them is more uniformly tempered before being moulded; but this advantage is partially neutralized by the presence of sand seams (§ 56). Machine-moulded soft-mud bricks rank next in compactness and uniformity of texture. Then come machine-moulded stiff-mud bricks, which vary greatly in durability with the kind of machine used in their manufacture. By some of the machines, the brick is moulded in layers (parallel to any face, according to the kind of machine), which are not thoroughly cemented, and which separate under the action of the frost. In compactness, the dry-clay brick comes last. However, the relative value of the products made by the different processes varies with the nature of the clay used.

3. The absorptive power is one of the most important elements, since it greatly affects the durability of the brick, particularly its resistance to the effect of frost (see §§ 31 and 32). Very soft, under-burned brick will absorb from 25 to 33 per cent. of their weight of water. Weak, light-red ones, such as are frequently used in filling in the interior of walls, will absorb about 20 to 25 per cent.; while the best brick will absorb only 4 or 5 per cent. A brick may be called good which will absorb not more than 10 per cent. See Table 9 (page 45).

4. The specific gravity of a brick does not indicate its quality, and depends mainly upon the amount of burning and the kind of fuel employed. Over-burned arch bricks, being both smaller and heavier than the better body bricks, have a considerably greater specific gravity, although inferior in quality.

5. The crushing strength is not a certain index of the value of a brick, although it is always one of the items determined in testing brick—if a testing-machine is at hand. For any kind of service, the durability of a brick is of greater importance than its ability to resist crushing,—the latter is only remotely connected with dura-

bility. Tests of the crushing strength of individual bricks are useful only in comparing different kinds of brick, and give no idea of the strength of walls built of such bricks (see § 246). Furthermore, the crushing strength can not be determined accurately, since it varies greatly with the size of the specimen and with the details of the experiments (see § 60).

6. Owing to both the nature of the quality tested and the facility with which such a test can be made, the determination of the transverse strength is one of the best means of judging of the quality of a brick. The transverse strength depends mainly upon the toughness of the brick,—a quality of prime importance in bricks used for paving, and also a quality greatly affecting the resistance to frost.

58. Absorbing Power. The less the amount of water absorbed by a brick the greater, in all probability, will be its durability. The amount of water absorbed is, then, an important consideration in determining the quality of a brick. There are different methods in use for determining the amount of water taken up by a brick, and these lead to slightly different results. Some experimenters dry the bricks in a hot-air chamber, while some dry them simply by exposing them in a dry room; some experimenters immerse the bricks in water in the open air, while others immerse them under the receiver of an air-pump; some immerse whole brick, and some use small pieces; and, again, some dry the surface with bibulous paper, while others allow the surface to dry by evaporation. Air-drying most nearly represents the conditions of actual exposure in masonry structures, since water not expelled in that way is in such a condition as not to do any harm by freezing. Immersion in the open air more nearly represents actual practice than immersion in a vacuum. The conditions of actual practice are best represented by testing whole brick, since some kinds have a more or less impervious skin. Drying the surface by evaporation is more accurate than drying it with paper; however, neither process is susceptible of mathematical accuracy.

The absorbing power given in Table 9, page 45, was determined by (1) drying whole brick in a steam-heated room for three weeks, (2) weighing and (3) immersing them in water for forty-four hours; and then (4) drying for four hours—until all the water on the surface was evaporated,—and, finally, (5) again weighing them.

The results in the table represent the mean of several observations. If the brick had been kiln-dried, or weighed before the surface water was entirely removed, the apparent absorption would have been greater.

Comparing the absorbing power of brick as given in the table on page 45 with that of stone on page 20, we see the absorbing power of the best brick is about equal to that of average limestone and sandstone, and much greater than marble and granite. For a method of rendering brick non-absorbent, see §§ 263–64.

59. Transverse Strength. The experiments necessary to determine the transverse strength of brick are easily made (§ 15), give definite results, and furnish valuable information concerning the practical value of the brick; hence this test is one of the best in use.

Table 6 gives the results of experiments made by the author on Illinois brick. The averages represent the results of from six to fifteen

TABLE 6.

Transverse Strength of Illinois Brick.

(Summarized from Table 9, page 45.)

Ref. No.	Kind of Brick.	Modulus of Rupture in Lbs. per Sq. In.*			Co-efficient of Transverse Strength.*		
		Max.	Min.	Average.	Max.	Min.	Aver.
1	Soft-clay, hand-moulded, —best 50% in kiln........	2,233	846	1,409	124	47	78
2	Soft-clay, machine-moulded,—best 50% in kiln....	2,354	1,135	1,712	142	63	95
3	Stiff-clay, machine-moulded,—best 50% in kiln....	1,475	764	1,114	82	42	62
4	Dry-clay (pressed).........	495	150	336	27	8	19
5	Secret Process............	4,348	2,235	3,217	241	124	178

experiments on brick from three localities. The "Max." and "Min." columns contain the average of the two highest and the two lowest results respectively.

The results in Table 7 were obtained under the direction of the Chief Engineer of the Lehigh Valley R. R. Each result represents

* For definition, see § 15.

the mean of seven to nine experiments on bricks from different localities. The results in Table 6 are considerably greater than

TABLE 7.

TRANSVERSE STRENGTH OF EASTERN BRICK.

Designation of Brick.	Modulus of Rupture in Lbs. per Sq. In.			Co-efficient of Transverse Strength.		
	Max.	Min.	Average.	Max.	Min.	Average.
Very hard..................	1,796	1,045	1,352	100	58	75
Hard......................	944	710	805	52	39	45
Medium....................	645	504	597	36	28	32
Soft......................	444	269	378	25	15	21

those in Table 7, the difference being due probably more to recent improvements in the manufacture of brick and to the method of selection than to locality. The brick from which the results in Table 6 were derived were obtained from manufacturers well known for the high quality of their products.

60. Crushing Strength. It has already been explained (§§ 7 to 14) that the results for the crushing strength of stone vary greatly with the details of the experiments; but this difference is even greater in the case of brick than in that of stone. In testing stone the uniform practice is to test cubes (§ 10) whose faces are carefully dressed to parallel planes. In testing brick there is no settled custom. (1) Some experimenters test half brick while others test whole ones; (2) some grind the pressed surfaces accurately to planes, and some level up the surfaces by putting on a thin coat of plaster of Paris, while others leave them in the rough; and (3) some test the brick set on end, some on the side, and others laid flatwise.

1. From a series of experiments* on soft brick, the author concludes that the crushing strength per square inch of a quarter of a brick is about *half* that of a whole one; and that a half brick is about *two thirds*, and *three quarters* of a brick about *five sixths*, as strong per square inch as a whole one; or, in other words, the strength of a quarter, a half, and three quarters of a brick, and a

* *Engineering News*, vol. xxi. p. 88.

whole one, are to each other as 3, 4, 5, and 6 respectively. The reason for this difference is apparent if a whole brick be conceived as being made up of a number of cubes placed side by side, in which case it is clear that the interior cubes will be stronger than the exterior ones because of the side support derived from the latter. For experiments showing the marked effect of this lateral support, see § 273. The quarter brick and the half brick have less of this lateral support than the whole one, and hence have correspondingly less crushing strength.

2. The strength of the specimen will vary greatly with the degree of smoothness of its bed-surfaces. To determine the difference between reducing the pressed surfaces to a plane and leaving them in the rough, the author selected six bricks of regular form and apparently of the same strength, and tested three in the rough and the other three after having reduced the pressed surfaces to planes by laying on a coating of plaster of Paris, which, after drying, was ground off to a plane. The amount of plaster remaining on the surfaces was just sufficient to fill up the depressions. Both sets were tested in a hydraulic press between cast-iron, parallel (self-adjusting), pressing surfaces. The average strength of those that were plastered was 2.06 times the strength of those that were not plastered. This difference will vary with the relative strength of the brick and the plaster. The average strength of the bricks whose surfaces were plastered was 9,170 pounds per square inch, which is more than that of the plaster used; and therefore it is highly probable that if the surfaces had been reduced to planes by grinding, the difference in strength would have been still greater. See also the last paragraph of § 9.

3. As before stated, some experimenters test brick flatwise, some edgewise, and some endwise. Since bricks are generally employed in such a position that the pressure is on the broadest face, it seems a little more satisfactory to lay the brick flatwise while testing it; but since the only object in determining the crushing strength of brick is to ascertain the relative strength of different bricks,—the crushing strength of the brick is only remotely connected with the crushing strength of the brick-masonry (§ 246),—the position of the brick while being tested is not a matter of vital importance. Doubtless the principal reason for testing them on end or edgewise is to bring them within the capacity of the testing-machine. However,

there is one good reason against testing brick flatwise; viz., all homogeneous granular bodies fail under compression by shearing along planes at about 45° with the pressed surfaces, and hence if the height is not sufficient to allow the shearing stresses to act freely, an abnormal strength is developed. See also § 10.

The relative strength of brick tested in the three positions—flatwise, edgewise, and endwise—varies somewhat with the details of the experiments; but it is reasonably well settled that the strength of homogeneous brick flatwise between steel or cast-iron pressing surfaces is one and a half to two times as much as when the brick is tested on end. A few experiments by the author * seem to indicate that the strength edgewise is a little more than a mean between the strength flatwise and endwise. If the brick is laminated (see paragraph 2, § 57), the relative strength for the three positions—flatwise, edgewise, and endwise—will vary greatly with the direction of the grain.

61. Comparatively few experiments have been made to determine the strength of brick, and they are far from satisfactory, since the manner of making the experiment is seldom recorded. The differences in the details of the experiments, together with the differences in the quality of the bricks themselves, are sufficient to cause a wide variation in the results obtained by different observers. The following data are given for reference and comparisons.

Rankine says that "strong red brick, when set on *end*, should require at least 1,100 lbs. per sq. in. to crush them; weak red ones, 550 to 800 lbs. per sq. in.; and fire bricks, 1,700 lbs. per sq. in." †

Experiments on the brick in general use in Berlin gave for "ordinary" brick, on *edge*, a strength of 2,930 lbs. per sq. in.; and for "selected" brick, 3,670 lbs. per sq. in. ‡

The brick used in the New York reservoir, when laid flat and packed with sand, showed an average strength, for four specimens, of 2,770 lbs. per sq. in.; and two samples tested between wood averaged 2,660 lbs. per sq. in. § Prof. Pike ‖ tested half brick flat-

* *Engineering News*, vol. xxi. p. 88.

† Civil Engineering, pp. 366 and 769.

‡ Van Nostrand's *Engineering Mag.*, vol. xxxiv. p. 240. From Abstracts of the Inst. of C. E.

§ Jour. Frank. Ins., vol. lxv. p. 333; also Trans. Am. Soc. of C. E., vol. ii. pp. 185–86.

‖ Jour. Assoc. Engineering Soc., vol. iv. pp. 366–67.

TABLE 8.

EXPERIMENTS ON BRICK WITH THE WATERTOWN TESTING-MACHINE.

Reference No.	Kind of Brick.	No. of trials.	Position while being tested.	Compressive strength in pounds per square inch.
1	Philadelphia,* Dobbin's, hard, machine-moulded	3	flatwise	9,846
2	" Excelsior, " " "	3	"	6,546
3	" Huhn's, " hand-moulded	5	"	14,558
4	" Dotterer's pressed	3	"	8,010
5	Washington,† pressed,—from four makers	4	"	7,092
6	" red, " three "	5	"	7,370
7	" arch, " " "	3	"	8,230
8	Boston,‡ face	3	"	13,925
9	" Bay State	3	"	11,406
10	" common	3	"	18,377
11	Common,§ dark red	1	"	18,990
12	" medium red	3	"	15,866
13	" light red	1	"	12,500
14	" dark red	3	endwise	14,050
15	" medium red	3	"	10,523
16	" light red	3	"	7,570
17	" medium red	3	a pile of three	3,723
18	Face,§	3	flatwise	13,353
19	" dark red	6	endwise	6,985
20	" " "	3	a pile of three	3,756

* Tested for the Commissioners for the Erection of Public Buildings. See Report of Commissioners, p. 32.
† Samples of bricks used in Pension Building. See Tests of Metals, etc., 1883, p. 290.
‡ Samples of bricks used in experimental brick piers (§ 246). See Tests of Metals, etc., 1883, pp. 217-19.
§ Samples of bricks used in experimental brick piers (§ 246). See Tests of Metals, etc., 1885, pp. 1138-61.

TABLE 9.

Absorptive Power, and Transverse and Compressive Strengths of Illinois Brick.

Reference No.	Kind of Brick.				Absorption. (See § 58.)	Transverse Strength. (See § 59.)		Compressive Strength. (See § 60.)		
	Condition of clay.	Method of moulding.	Trade name.	Proportion of kiln represented by the brick tested.	Air-dried 3 weeks, immersed 44 hours, and air-dried 4 hours.	Modulus of rupture,* lbs. per square inch.	Co-efficient of breaking strength.*	Average dimensions of bed-area, inches.	Condition of pressed surfaces.	Crushing strength, lbs. per square inch.
1	soft	hand	soft	15%	21.05	148	8	....	not plastered	674
2	"	"	face	10	19.2	....	....	4 × 4¼	"	1,835
3	"	"	paving	50	5.8	1,306	70	2½ × 3½	plastered	11,340
4	"	"	arch	25	....	1,057	58	....	....	....
5	soft	hand	best	....	2.0	2,304	129	....	....	....
6	"	"	hard	....	1.1	1,817	78	....	....	....
7	"	"	paving	60	4.8	1,429	79	2¼ × 4	plastered	6,190
8	"	"	"	60	5.6	1,117	68	2½ × 2½	"	4,800
9	soft	machine	face	20	9.0	968	53	3¼ × 3¾	not plastered	4,811
10	"	"	paving	50	4.5	1,712	95	2 × 3½	plastered	13,085†
11	"	"	arch	30	3.2	851	47	....	....	....
12	stiff	machine	....	50	6.3	996	55	3 × 3¼	not plastered	3,486
13	dry	machine	face	....	10.7	260	14	3¾ × 4	"	2,719
14	"	"	"	....	15.0	418	23	3¾ × 4½	"	3,578
15	dry	machine	face	....	2.0	3,217	178	....	....	‡

General Explanations.—Identically the same brick were tested for absorbing power, for transverse strength, and for crushing strength. Each r nsverse strength represents the mean for seven to ten trials, and each result for crushing strength the mean ...

Nos. 1-4 and 5-8. These bricks are alike, excep

Nos. 3 and 10. Bricks of these brands are givin durability, for these experiments.

Nos. 7 and 8. These brick were taken out of a r durability for seventeen years.

No. 13. These brick were taken from the face

* For definition, see § 15. † Not plastered,—6,190. ‡ A whole brick, on end, showed no signs of crushing under 11,088 pounds per square inch.

wise between sheets of pasteboard with the following results: St. Louis brick, 6,417 lbs. per sq. in. (the average of six trials); and pressed brick, 2,519 lbs. per sq. in. (the average of thirteen samples from ten localities).

The results in Table 8 (page 44) were made with the U. S. testing-machine at the Watertown (Mass.) Arsenal.* In each experiment the pressed surfaces were "carefully ground flat and set in a thin facing of plaster of Paris, and then tested between steel pressing surfaces."

The experiments given in Table 9 (page 45) were made by the author, on Illinois brick. The bricks were crushed between self-adjusting cast-iron pressing surfaces. Although No. 11 shows an average absorption, a moderate transverse strength, and a high crushing strength, this particular brand of brick disintegrated rapidly by the frost. This is characteristic of this class of brick, and is caused by the clay's being forced into the moulds or through the die in such a way as to leave the brick in laminæ, not well cemented together. A critical examination of the brick with the unaided eye gave no indication of a laminated structure, and yet compressing the brick in two positions—sidewise and edgewise—never failed to reveal such structure. The crushing strength in the table was obtained when the pressure was applied to the edges of the laminæ. In experiments Nos. 12, 13, and 14 the pressed surfaces were so nearly mathematical planes that possibly these bricks stood more than they would have done if their beds had been plastered. The strength of No. 15 was beyond the capacity of the machine; a whole brick, on end, stood 11,083 lbs. per sq. in. without any cracks or snapping sounds—which usually occur at about half of the ultimate strength.

62. Size and Weight. In England the legal standard size for brick is $8\frac{3}{4} \times 4\frac{3}{8} \times 2\frac{3}{4}$ inches. In Scotland the average size is about $9\frac{1}{2} \times 4\frac{1}{2} \times 3\frac{1}{2}$ inches; in Germany, $9\frac{7}{8} \times 4\frac{3}{4} \times 2\frac{5}{8}$ inches; in Austria, $11\frac{1}{8} \times 5\frac{1}{2} \times 2\frac{5}{8}$ inches; in Cuba, $11 \times 5\frac{1}{2} \times 2\frac{5}{8}$ inches; and in South America, $12\frac{3}{4} \times 6\frac{1}{4} \times 2\frac{1}{2}$ inches.

In the United States there is no legal standard, and the dimensions vary with the maker. In the Eastern States $8\frac{1}{4} \times 4 \times 2\frac{1}{4}$ inches is a common size for brick, of which 26 make a cubic foot; but in the West the dimensions are usually a little smaller. The National Brick-makers' Association in 1887 and the National

* Compiled from the annual reports for 1883–85.

Traders and Builders' Association in 1889 adopted $8\frac{1}{4} \times 4 \times 2\frac{1}{4}$ inches as the standard size for common brick, and $8\frac{3}{8} \times 4\frac{1}{8} \times 2\frac{1}{4}$ for face brick. The price should vary with the size. If, reckoned according to cubic contents, brick $8 \times 4 \times 2$ inches is worth $10 per thousand, brick $8\frac{1}{4} \times 4\frac{1}{4} \times 2\frac{1}{4}$ is worth $12.33 per thousand, and $8\frac{1}{2} \times 4\frac{1}{2} \times 2\frac{1}{2}$ is worth $15 per thousand. Further, where brick is laid by the thousand, small bricks are doubly expensive. Since bricks shrink in burning, in proportion to the temperature to which they are exposed, the amount differing with the different kinds of clays, it is impossible to have the size exactly uniform. Re-pressed and machine-moulded bricks are more nearly uniform in size than hand-moulded.

The size of brick and the thickness of the mortar joint should be such that brick may be laid flat, edgewise, or set vertically, and still fit exactly. These proportions are seldom realized.

Re-pressed brick weighs about 150 lbs. per cu. ft.; common hard brick, 125; inferior, soft brick, 100. Common bricks will average about $4\frac{1}{2}$ lbs. each.

63. Cost. Brick is sold by the thousand. At Chicago, in 1887, the "best sewer" brick cost $9; common brick, from $6 to $7.

CHAPTER III.

LIME AND CEMENT.

64. Classification. Considered as materials for use in the builder's art, the products derived from the calcination of pure and impure limestones are classified as common or fat lime, hydraulic lime, and hydraulic cement. Common lime is sometimes called air-lime, because a paste or mortar made from it requires exposure to the air to enable it to "set," or harden. The hydraulic limes and cements are also called water-limes and water-cements, from their property of hardening under water.

Common lime is used in making the mortar for most architectural masonry, and until recently it was generally employed in engineering masonry; but the opinion is rapidly gaining ground that only cement mortar should be employed in engineering structures requiring great strength or subject to shock. On most first-class railroads hydraulic cement mortar is used in all masonry structures. This change in practice is largely due to the better appreciation of the superiority of hydraulic cement as a building material. Although it has been manufactured for about fifty years, the amount used was comparatively limited until within the last twenty years. At present large quantities are imported from Europe, and very much more is made in this country. Hydraulic lime is neither manufactured nor used in this country.

The following discussion concerning common and hydraulic limes is given as preliminary to the study of hydraulic cements rather than because of the importance of these materials in engineering construction.

Art. 1. Common Lime.

65. Description. The limestones which furnish the common lime are seldom, if ever, pure; but usually contain, besides the carbonate of lime, from 3 to 10 per cent. of impurities,—such as silica,

alumina, magnesia, oxide of manganese, and traces of the alkalies. Lime—variously designated as common lime, quicklime, or caustic lime—is a protoxide of calcium, and is produced when marble, or any other variety of pure or nearly pure carbonate of lime, is calcined with a heat of sufficient intensity and duration to expel the carbonic acid. It has a specific gravity of 2.3, is amorphous, highly caustic, has a great avidity for water, and when brought into contact with it will rapidly absorb nearly a quarter of its weight of that substance. This absorption is accompanied and followed by a great elevation of temperature, by the evolution of hot and slightly caustic vapor, by the bursting of the lime into pieces; and finally the lime is reduced to a powder, the volume of which is from two and a half to three and a half times the volume of the original lime—the increase of bulk being proportional to the purity of the limestones. In this condition the lime is said to be slaked, and is ready for use in making mortar.

The paste of common lime is unctuous and impalpable to sight and touch; hence these limes are sometimes called fat or rich limes, as distinguished from others known as poor or meager limes. These latter usually contain more or less silica and a greater proportion of other impurities than the fat limes. In slaking they exhibit a more moderate elevation of temperature; evolve less vapor; are seldom reduced to an impalpable homogeneous powder; yield thin paste; and expand less. They are less valuable for mortar than the fat limes, but are extensively employed as fertilizers. When used for building purposes they should, if practicable, be reduced to powder by grinding, in order to remove all danger of subsequent slaking.

66. TESTING. Good lime may be known by the following characteristics: 1. Freedom from cinders and clinkers, with not more than 10 per cent. of other impurities,—as silica, alumina, etc. 2. Chiefly in hard lumps, with but little dust. 3. Slakes readily in water, forming a very fine smooth paste, without any residue. 4. Dissolves in soft water, when this is added in sufficient quantities. These simple tests can be readily applied to any sample of lime.

67. PRESERVING. As lime abstracts water from the atmosphere and is thereby slaked, it soon crumbles into a fine powder, losing all those qualities which render it of value in building. On this account great care must be taken that the lime to be used is freshly

burned, as may be known by its being in hard lumps rather than in powder. Lime is shipped either in bulk or in casks. If in bulk, it is impossible to preserve it for any considerable time; if in casks, it may be preserved for some time by storing in a dry place.

Common lime, when mixed to a paste with water, may be kept for an indefinite time in that condition without deterioration, if protected from contact with the air so that it will not dry up. It is customary to keep the lime paste in casks, or in the wide, shallow boxes in which it was slaked, or heaped up on the ground, covered over with the sand to be subsequently incorporated with it in making mortar. It is convenient for some purposes to keep the slaked lime on hand in a state of powder, which may be done in casks under cover, or in bulk, in a room set apart for that purpose. The common limes contain impurities which prevent a thorough, uniform, and prompt slaking of the entire mass, and hence the necessity of slaking some days before the lime is to be used, to avoid all danger to the masonry by subsequent enlargement of volume and change of condition.

A paste or mortar of common lime will not harden under water, nor in continuously damp places excluded from contact with the air. It will slowly harden in the air, from the surface toward the interior, by desiccation and the gradual absorption of carbonic-acid gas, by which process a subcarbonate with an excess of hydrated base is formed.

68. Cost. Lime is sold by the barrel (about 230 pounds net), or by the bushel (75 pounds). At Chicago the average price, in 1887, was from 55 to 65 cents per barrel.

Art. 2. Hydraulic Lime.

69. Description. Hydraulic lime is like common lime in that it will slake, and differs from it in that it will harden under water. Hydraulic lime may be either argillaceous or siliceous. The former is derived from limestones containing from 10 to 20 per cent. of clay, homogeneously mixed with carbonate of lime as the principal ingredient; the latter from siliceous limestones containing from 12 to 18 per cent. of silica. Small percentages of oxides of iron, carbonates of magnesia, etc., are generally present.

During the burning, the carbonic acid is expelled, and the silica and alumina entering into combination with a portion of the lime

form both the silicate and the aluminate of lime, leaving in the burnt product an excess of quick or caustic lime, which induces slaking, and becomes hydrate of lime when brought into contact with water. The product owes its hydraulicity to the crystallizing energy of the aluminate and the silicate of lime.

70. Classification. Argillaceous hydraulic limes are arranged in three classes, according to their amount of hydraulic energy:

1. Feebly hydraulic—containing 10 to 20 per cent. of impurities. This slakes in a few minutes, with crackling, heat, and emission of vapor. If made into a paste and immersed in water in small cakes, in from 12 to 15 days it will harden so as to resist crushing between the thumb and finger.

2. Ordinary hydraulic—containing 17 to 24 per cent. of impurities. Slakes after an hour or two, with slight heat and fumes, without crackling. Sets under water in 6 or 8 days.

3. Eminently hydraulic—containing at least 20 per cent. of impurities. Slakes very slowly and with great difficulty, with slight heat. Sets under water in 12 to 20 hours, and becomes hard in 2 to 4 days.

Artificial hydraulic lime can be manufactured by mixing together, in suitable proportions, thoroughly slaked common lime and unburnt clay, tempering the mixture with water, and then burning it in the form of bricks or rounded balls in an ordinary lime-kiln. The burnt material can be slaked in the ordinary way. For the common lime, powdered limestone may be substituted. It is better, however, when it becomes necessary to resort to artificial mixtures to produce the hydraulic ingredient of mortar, to make hydraulic cement, on account of its superior hydraulic energy.

No hydraulic lime is manufactured in the United States. It is manufactured in several localities in France, notably at Teil and Scilly, from which places large quantities were formerly brought to this country.

Hydraulic lime is usually slaked, screened, and packed in sacks or barrels before being sent to market. The screening takes out a lumpy unslaked residue, which is often ground and mixed with lime. This residue is sometimes natural hydraulic cement, in which case it improves the quality of the lime. Sometimes it is unburned clay, which constitutes an adulteration. The use of these unslaked lumps requires watchfulness, in order that the in-

troduction of ingredients that are either worthless or dangerous may be avoided.

ART. 3. HYDRAULIC CEMENT.

71. CLASSIFICATION. Hydraulic cements may be divided into three classes, viz., Portland, Rosendale, and Pozzuolana. The first two differ from the third in that the ingredients of which the former are composed must be roasted before they acquire the property of hardening under water, while the ingredients of the latter need only to be pulverized and mixed with water to a paste. The first two classes include the ordinary hydraulic cements, and the third class includes the pozzuolanas. The former are by far the more valuable, and since the discovery of the ordinary hydraulic cements pozzuolana is never used.

72. Portland cement is heavy, slow-setting, and has great ultimate strength. Rosendale is light, quick-setting, and has less ultimate strength. Roughly speaking, the second weighs about two thirds as much as the first, sets in one tenth of the time, and attains to about half the ultimate strength of the Portland. There is a great variety of brands of each class, which differ from each other in minor particulars.

Pozzuolana cement is so called from Pozzuoli, Italy, near which place the ingredients of which it is composed were first found. Portland cement derives its name from the resemblance which hardened mortar made of it bears to a stone found in the isle of Portland, off the south coast of England. There is a great diversity and some confusion in the names employed to designate that cement which above is called Rosendale. The terms "natural," "American," and "Rosendale" are frequently used. *It is here proposed to call this cement* ROSENDALE, *from the place—Rosendale, Ulster Co., N. Y.—where it was first made.*

In Europe the term Roman cement is used to designate a natural cement of about the same character as Rosendale. This cement was first known as Parker's, and was afterwards called Roman, probably on the supposition, or to convey the impression, that it was the cement employed by the early Romans; but it is in no way similar to their cement. They seem to have been ignorant of the art of conferring hydraulicity upon a limestone by roasting it. Vicat, of France, discovered this principle in 1818, although

Roman cement had been made by a tentative process as early as 1796. Portland cement was made first in England about 1843, in America about 1874; Rosendale was first made in the United States about 1837. A small quantity of fancy cements, known as Parian, Martin's, Keene's, etc., are imported for use in the production of terra-cotta trimmings, imitation marble, for extra hard finish on walls, etc.

73. Portland Cement. The natural stone from which the original Portland cement is derived contains from 20 to 22 per cent. of clay and 78 to 80 per cent. of carbonate of lime. The clay itself is composed of 1½ to 2 parts of silica to 1 of alumina. When calcined at a high, long-continued heat, all or nearly all the silica and alumina of the clay combines with a portion of the lime; consequently the burned product does not contain any uncombined, and therefore inert, silica and alumina to adulterate the cement and impair its hydraulic properties. As the quantity of uncombined lime is not sufficient to cause the mass to slake to powder in the presence of water, the cement must be reduced to powder by grinding between ordinary millstones.

The superiority of Portland cement appears to depend in great measure upon the presence of the double silicate of lime and alumina, which is formed only at a high heat. The weight of Portland cement, as well as its hydraulic energy and its ultimate strength and hardness, is increased by augmenting (within the limit of vitrification) the intensity and duration of the heat employed in burning. The initial hydraulic activity, however, is diminished by high burning, so that the best Portland cements are slowest in setting. A cement weighing 100 pounds to the bushel may be burnt so as to weigh 125 pounds to the bushel, and its strength will be nearly doubled thereby.

74. *Artificial Portland.* Fully nineteen twentieths of all the Portland cement used at the present day is artificial. It is made by thoroughly mixing together, in suitable proportions, clay and finely pulverized carbonate of lime (either chalk, marl, or compact limestone), burning the mixture in kilns at a high heat, and then grinding the burnt product to fine powder between ordinary millstones.

In England the ingredients of the cement are mixed together with a large body of water, and afterwards dried, burned, and ground. This is called the "wet process." In Germany the in-

gredients are mixed dry. Each process is thought to be well adapted to the character of the materials employed.

Artificial Portland is made at a number of places in this country, but to much greater extent at Allentown and Egypt, Penn., than anywhere else. The brand manufactured at Allentown, "Saylor's," is nearly, if not quite, equal to the best imported Portland, and has been extensively used in the last few years. The Portland-cement industry in this country is in its infancy, the first factory having been established in 1875. In the past few years other factories have been built in different parts of the country. In 1886 we imported 650,000 and made 150,000 barrels of Portland cement.* The raw material suitable for the manufacture of Portland cement exists in great abundance in nature, and with proper care a high-class Portland cement might be produced in almost any part of the country.

75. Rosendale Cement.† By far the greater part ‡ of the hydraulic cement used in the United States is of the Rosendale type. It is made from the argillo-magnesian limestones, that is, limestones of which the principal ingredients are carbonate of lime, carbonate of magnesia, and clay. The process followed in their manufacture is essentially the same as for Portland cement. The stone is quarried, broken into pieces, and burned in a kiln. The burnt cement is then crushed into small fragments, ground between ordinary millstones, packed, and sent to market.

A light, quick-setting cement can also be made from an argillaceous limestone containing more than 23 per cent. of clay homogeneously mixed through the mass. Any magnesian limestone containing as high as 60 per cent. of carbonate of magnesia may be presumed to be capable of yielding hydraulic cement of greater or less value, if properly burned, no matter whether clay be present or not. Dolomite, or the double carbonate of lime and magnesia, when burned at a low heat, reduced to powder, and made into mortar, also exhibits hydraulic properties. Pure carbonate of magnesia, when burned at a moderate intensity, ground to fine powder, and made into paste with sea-water, makes a cement which is superior in hardness and strength to any other, not excepting even Portland cement.

* U. S. Bureau of Mining Statistics, 1886.

† See § 72.

‡ In 1886, according to the U. S. Bureau of Mining Statistics, it was 85 per cent.

76. The natural bed of limestones from which the original Rosendale is made, covers about one third the State of New York, and the western part of Vermont, besides extending in a belt through New Jersey, Pennsylvania, Maryland, Virginia, and East Tennessee. Deposits suitable for the manufacture of natural cements are found at numerous other points in this country. Is it too much to assume that there is no very large area in which a stone can not be found from which some grade of natural cement can be made?

Cement of the Rosendale type is made in large quantities in Ulster County, and at Akron and Buffalo, N. Y.; at Sandusky, Ohio; Louisville, Ky.; Utica, Ill.; Milwaukee, Wis.; Kansas City, Mo.; Trinidad, Col.; Kensington, Conn.; and possibly at other places.

The total production of cement of the Rosendale type in the United States in 1886 was 4,200,000 barrels, of which nearly half was produced in Ulster County, N. Y.

77. Pozzuolana. This is not a very important kind of cement, since the ingredients are not widely distributed, and also because the strength of the cement is much less than that of other hydraulic cements.

The substance first known to possess the peculiar property of hydraulicity was volcanic ashes; and the discovery was made at Pozzuoli, near the base of Mount Vesuvius,—hence the name. Vitruvius and Pliny both mention that pozzuolana was extensively used by the Romans before their day; and Vitruvius gives a formula for its use in monolithic masonry, which with slight variations has been followed in Italy ever since. It is as follows: "12 parts pozzuolana, well pulverized; 6 parts quartzose sand, well washed; and 9 parts rich lime, well slaked." This constitutes a mortar, which may be used either as ordinary mortar or as a matrix for filling in.

"Trass, a volcanic earth, closely resembles pozzuolana, and is employed substantially in the same way. It is found on the Rhine between Mayence and Cologne, and in various localities in Holland. The *arènes* are a species of ocherous sand containing so large a proportion of clay that they can be mixed into a paste with water without the addition of lime, and used in that state for common mortar. Mixed with rich lime they yield hydraulic mortars of

considerable energy. Many of the natural pozzuolanas are improved by a slight roasting, and an artificial pozzuolana may be produced by subjecting clay to a slight calcination."

Brick dust mixed with common lime produces a feebly hydraulic mortar. Forge scales from smiths' anvils and the slags from iron foundries are artificial pozzuolanas.

ART. 4. TESTS OF CEMENT.*

78. The value of cements varies greatly with their physical properties; and since one lot of cement is liable to differ very much from another lot of the same brand, it is very necessary to be able to test the character of any particular cement. The properties of a cement which are usually examined to determine its constructive value are: (1) color, (2) weight, (3) activity, (4) soundness, (5) fineness, and (6) strength. The last three are the most important.

79. COLOR. The absolute color of a cement indicates but little, since it is chiefly due to oxides of iron and manganese, which in no way affect the cementitious value; but for any given kind variations in shade may indicate differences in the character of the rock or in the degree of burning. With Portland cement, gray or greenish gray is generally considered best; bluish gray indicates a probable excess of lime; and brown, an excess of clay. An undue proportion of under-burned material is generally indicated by a yellowish shade, with a marked difference between the color of the hard-burned, unground particles retained by a fine sieve and the finer cement which passes through the sieve.

American cements are generally brown, in light or dark shades. In Rosendale cement a light color generally indicates an inferior, under-burned rock.

80. WEIGHT. For any particular cement the weight varies with the degree of heat in burning, the degree of fineness in grinding, and the density of packing. Other things being the same, the harder-burned varieties are the heavier. The finer a cement is ground the more bulky it becomes, and consequently the less it weighs. Hence light weight may be caused by laudable fine grinding or by objectionable under-burning.

The weight per unit of volume is usually determined by sifting

* Contributed to *Engineering News*, vol. xv. pp. 283–84.

the cement into a measure as lightly as possible, and striking the top level with a straight-edge. In careful work the height of fall is specified. For a method of still greater refinement, see Transactions of the American Society of Civil Engineers, vol. xiii. p. 55. Since the cement absorbs moisture, the sample must be taken from the interior of the package. The weight per cubic foot is neither exactly constant, nor can it be determined precisely; and for the practical purpose of the user is of very little service in determining the value of a cement. However, it is often specified as one of the requirements to be fulfilled.

The following values,* determined by sifting the cement with a fall of three feet into a box having a capacity of one tenth of a cubic foot, may be taken as fair averages for ordinary cements. The difference in weight for any particular kind is mainly due to a difference in fineness.

Portland, English, and German, .	77 to 90 lbs. per cubic foot.
" fine-ground French, . .	69 " " " "
" American, ...	95 " " " "
Roman, ...	54 " " " "
Rosendale, ...	49 to 56 " " " "
Lime of Teil, ...	50 " " " "

Since a bushel is 1.244 cubic feet, the weight per bushel can be obtained sufficiently close by adding 25 per cent. to the above quantities. However, it is better to make the cubic foot the standard unit measure.

81. Activity. A mortar is said to have *set* when it has attained such a degree of induration that its form can not be altered without causing a fracture, *i.e.*, when it has entirely lost its plasticity. Some cements set quickly, while others are comparatively slow in developing the first indications of hydraulicity. This property is called hydraulic quickness or activity. A quick-setting cement is especially valuable in constructions under water.

A distinction should be carefully made between hydraulic activity and hydraulic energy or strength. The former refers to the time required to attain a small degree of strength, and the latter to the amount of strength ultimately attained. There is no necessary relation between time of setting and ultimate strength; but, as a

* Transactions of the American Society of Civil Engineers, vol. xiv. p. 148.

general rule, the slow-setting cements ultimately attain to a greater strength than quick-setting ones.

"The effects of a variation of temperature upon the hydraulic quickness of mortars—whether derived from hydraulic lime, hydraulic cement, a mixture of common lime and pozzuolana, or produced by artificial means—is very marked: so much so, indeed, that in all comparative tests of this kind it is important to adopt some fixed standard of temperature, not only for the water with which the cement is mixed, as well as that in which the cement is immersed, but for the dry ingredients, and the surrounding atmosphere. All cements are not equally sensitive to a variation of temperature."

The activity of cement may be increased by adding a quicker-setting cement,—as plaster of Paris,—lime, clay, or even grease. All such ingredients, particularly the last, weaken the resulting mortar.

82. Test of Activity. "To test hydraulic activity, mix cement with 25 to 30 per cent. of its weight of clean water,* having a temperature of between 65° F. and 70° F., to a stiff plastic mortar, and make one or two cakes or pats 2 or 3 inches in diameter and about ½ inch thick. As soon as the cakes are prepared, immerse in water at 65° F., and note the time required for them to set hard enough to bear respectively a $\frac{1}{12}$-inch wire loaded to weigh ¼ pound, and a $\frac{1}{24}$-inch wire loaded to weigh 1 pound. When the cement bears the light weight, it is said to have begun to set; when it bears the heavy weight, it is said to have entirely set." Cements, however, will increase in hardness long after they can just bear the heavy wire. The activity of the cement is measured by the time which elapses between the time when the first weight is supported and that when the second is just borne. Notice that with the wires as above the weight per unit of surface in the second case is sixteen times as much as in the first. Hence it is not necessary to have the diameters as stated, but only to have the pressure per unit of area sixteen times as much in the one case as in the other; the same wire may be used in both tests, the load only being varied.

Different kinds and brands of cement vary greatly in the time required to set. Some brands of Rosendale cement will support

* The water required to make a stiff paste will vary somewhat with the kind and freshness of the cement. On the average, neat Portland cement mortar requires about 25 per cent. of its weight of water, and neat Rosendale about 30 per cent.; for 1 part cement to 1 part sand, about 15 per cent. of the total weight of the sand and cement; and for 1 cement to 3 sand, about 12 per cent.

the heavy wire in two minutes, and some brands of Portland in not less than 12 hours. Cold retards the setting. Freshly-ground cements set quicker than older ones. The quick-setting cements usually set so that experimental samples can be handled within 5 to 30 minutes after mixing; the slow-setting cements require from 1 to 8 hours.

83. SOUNDNESS. Soundness refers to the property of not expanding or contracting, or cracking or checking in setting. These effects may be due to free lime, free magnesia, or to unknown causes. Testing soundness is therefore determining whether the cement contains any *active* impurity. An inert adulteration or impurity affects only its economic value; but an active impurity affects also its strength and durability.

Soundness may be tested as follows: Immerse in water a small pat of neat cement mortar with thin edges (§ 82), and examine it from day to day to see if it becomes contorted or if cracks show themselves at the edges. If there is cracking or contortion (sometimes called "blowing"), it is due to the hydration and consequent expansion of the lime or magnesia. If the effect is due to lime, the cement can be improved by exposure to the air, thus allowing the free lime to slake. This treatment is sometimes called "cooling the cement." The presence of uncombined magnesia is more harmful than that of lime. A number of important masonry structures have failed in recent years owing to the expansion caused by the hydration of the magnesia in the cement employed in their construction.

It is generally held that 1½ or 2 per cent. of uncombined magnesia in Portland cement is dangerous, particularly in damp places. The best Portland cements have very much less than this. Several standard brands contain only 0.46 to 1.27 per cent. of magnesia.

Since cement of the Rosendale type is made of magnesian limestone, it contains from 5 to 20 per cent. of magnesia. Chemists are not agreed as to the manner in which the different constituents are combined, and consequently are not agreed either as to the amount or effect of free magnesia in such a cement. Fortunately, it is not necessary to resort to a chemical analysis to determine the amount of free lime or free magnesia present, for a cement which successfully stands the above test for soundness for one or two

days can be used with confidence. The time required to test soundness can be shortened, or the value of the test increased, by keeping the water in which the pat is immersed at, say, 100° F.

The soundness of a cement may also be tested by placing some mortar in a glass tube (a swelled lamp chimney is excellent for this purpose), and pouring water on top. If the tube breaks, the cement is unfit for use in damp places. A less delicate and less valuable test than either of the above is to note whether the cement heats when mixed with water. A thermometer is sometimes used in making this test.

Some idea of the quality of cement may be gained by exposing to the air a small cake of neat cement mortar and observing its color. "A good cement should be uniform bluish gray throughout, yellowish blotches indicating poor cement. The color of the cement, when left in the air, indicates the quality much better than when the cement is put into water." This test is not generally considered of much value.

Tests of soundness should not only be carefully conducted, but should extend over considerable time. Occasionally cement is found * which seems to meet the usual tests for soundness, strength, etc., and yet after considerable time loses all coherence and falls to pieces.

84. FINENESS. The question of fineness is wholly a matter of economy. Cement, until ground, is a mass of partially vitrified clinker, which is not affected by water, and which has no setting power. It is only after it is ground that the addition of water induces crystallization. Consequently the coarse particles in a cement have no setting power whatever, and may for practical purposes be considered only as so much sand and essentially an adulterant.

There is another reason why it should be well ground. A mortar or concrete being composed of a certain quantity of inert material bound together by a cementing material, it is evident that to secure a strong mortar or concrete it is essential that each piece of aggregate shall be entirely surrounded by the cementing material, so that no two pieces are in actual contact.

Obviously, then, the finer a cement the greater surface will a

* Trans. Am. Soc. of C. E., vol. xiii. p. 64, and vol. xiv. p. 149.

given weight cover, and the more economy will there be in its use.

Fine cement can be produced by the manufacturers in three ways: 1, by supplying the millstone with comparatively soft, under-burnt rock, which is easily reduced to powder; 2, by running the stones more slowly, so that the rock remains longer between them; or, 3, by bolting through a sieve and returning the unground particles to the stones. The first process produces an inferior quality of cement, while the second and third add to the cost of manufacture.

It is possible to reduce a cement to an impalpable powder, but the proper degree of fineness is reached when it becomes cheaper to use more cement in proportion to the aggregate than to pay the extra cost of additional grinding.

85. Measuring Fineness. The degree of fineness of a cement is determined by measuring the per cent. which will not pass through sieves of a certain number of meshes per square inch. The Committee of the American Society of Civil Engineers* recommend the determination "by weight of the per cent. that is rejected by sieves of 2,500, 5,476, and 10,000 meshes to the square inch respectively, the first-mentioned sieve being of No. 35, the second of No. 37, and the third of No. 40 wire gauge." These sieves are usually referred to by the number of meshes per linear inch; the first being known as No. 50, the last as No. 100. It is stated † that, as sold, the number of meshes varies somewhat, and that the number of wires is generally less, by about 10 per cent., than the number of the sieve. The diameter of the holes is about equal to the diameter of the wire.

German Portland cements are commonly ground finer than English. "Most English manufacturers grind their cement to such a degree of fineness that when sifted through a sieve having 2,500 holes (50 by 50) to the square inch, it shall leave a residue of not more than 10 per cent. by weight. Cement ground to this fineness will leave from 19 to 20 per cent. of residue on a 4,900 (70 by 70) sieve, and practically nothing on a 625 (25 by 25) sieve." This is supposed to be the most economical degree of fineness.

* Trans., vol. xiii. p. 54.
† Trans. Am. Soc. of C. E., vol. xiv. p. 144.

Different brands of Rosendale cement vary considerably in their fineness. Those of the best reputation will leave from 4 to 10 per cent. residuum on the No. 50 sieve; other brands, from 10 to 23 per cent.

86. Strength. The strength of cement mortar is usually determined by submitting a specimen of known cross section to a tensile strain. The reason for adopting tensile tests is that comparatively light strains produce rupture; and that, since mortar is less strong in tension than in compression, in most cases of failure of mortar it is broken by tensile stress, even though the masonry be under compression (§ 9).

87. The Testing-machine. The details of the form of the specimen to be tested (the briquette) as recommended by the Committee of the American Society, are given in Fig. 2. The method of placing the briquette in the machine is shown in Fig. 3. In

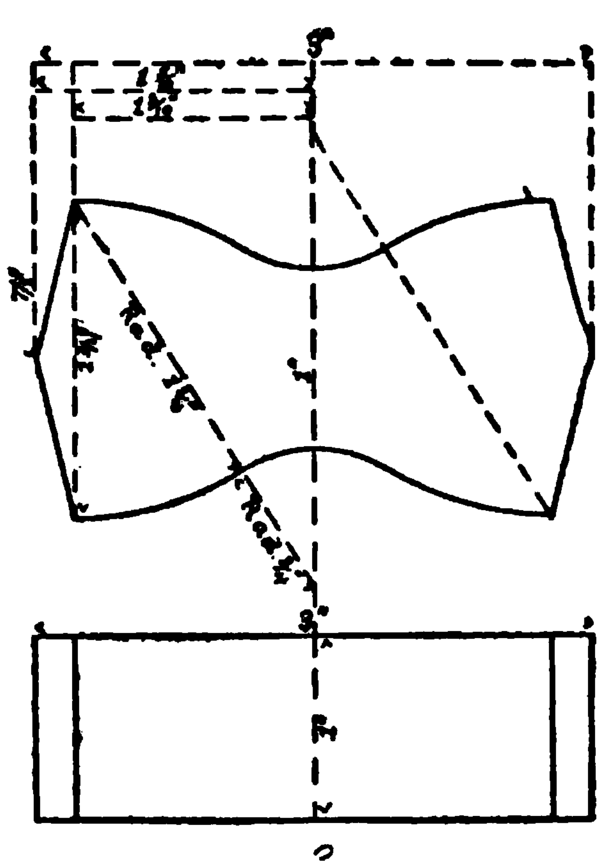

Fig. 2.

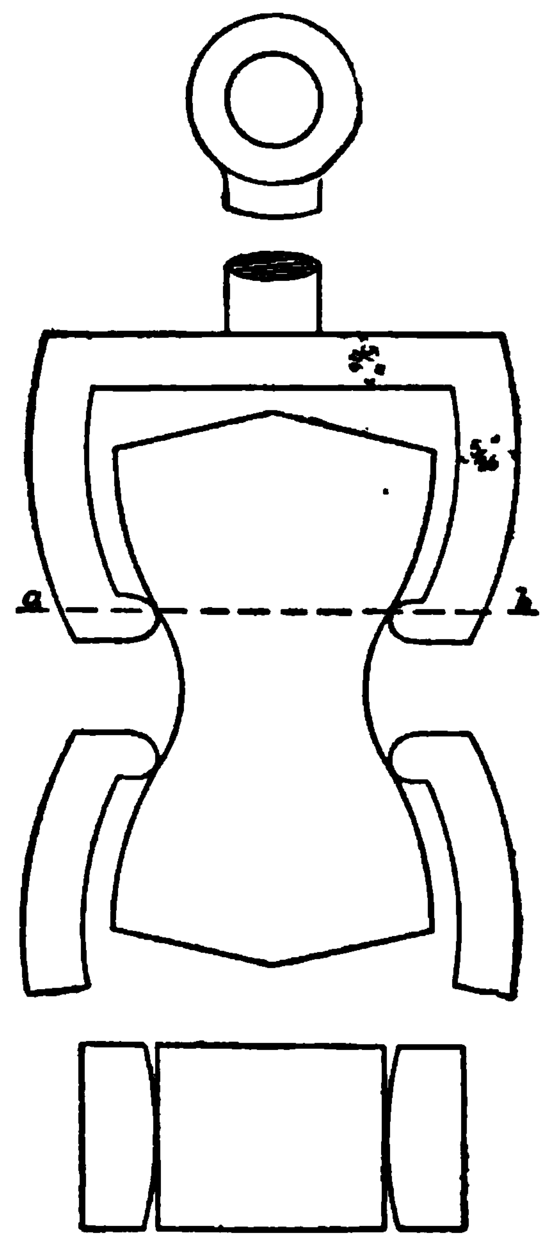

Fig. 3.

applying the stress, it is also recommended to make the initial strain 0, and increase it regularly at the rate of 400 lbs. per minute

until rupture takes place. "For a weak mixture one half the speed is recommended." *

There are many machines on the market, made specially for testing the strength of cement. Fig. 4 represents a cement-testing machine which can be made by an ordinary mechanic at an expense of only a few dollars. Although it does not have the conveniences and is not as accurate as the more elaborate machines, it is valuable where the quantity of work will not warrant a more expensive one, and in many cases is amply sufficient. It was devised by F. W. Bruce for use at Fort Marion, St. Augustine, Fla., and reported to the *Engineering News* † by Lieutenant W. M. Black, U. S. A.

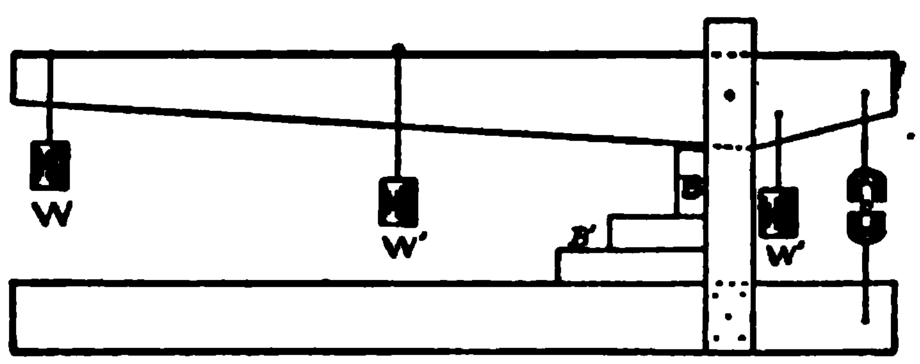

FIG. 4.

W, fixed weight. *W'*, rolling weight. *W''*, counterpoise. *B'*, block for shearing. *B*, block for crushing. *C*, tensile strain clips.

The machine consists essentially of a counterpoised wooden lever 10 feet long, working on a horizontal pin between two broad uprights 20 inches from one end. Along the top of the long arm runs a grooved wheel carrying a weight. The distances from the fulcrum in feet and inches are marked on the surface of the lever. The clamp for holding the briquette for tensile tests is suspended from the short arm, 18 inches from the fulcrum. Pressure for shearing and compressive stresses is communicated through a loose upright, set under the long arm at any desired distance (gen-

* The following data are given by H. Faija, an English authority, as showing the effect of a variation in the speed of applying the strain :

Rate.	Tensile Strength.
100 pounds in 120 seconds	400 pounds.
100 " " 60 "	415 "
100 " " 30 "	430 "
100 " " 15 "	450 "
100 " " 1 "	493 "

† Vol. v. pp. 194–96.

erally 6 or 12 inches) from the fulcrum. The lower clip for tensile strains is fastened to the bed-plate. On this plate the cube to be crushed rests between blocks of wood, and to it is fastened an upright with a square mortise at the proper height for blocks to be sheared. The rail on which the wheel runs is a piece of light T-iron fastened on top of the lever. The pin is iron, and the pin-holes are reinforced by iron washers. The clamps are wood, and are fastened by clevis joints to the lever arm and bed-plate respectively. When great stresses are desired, extra weights are hung on the end of the long arm. Pressures of 3,000 pounds have been developed with this machine.

For detailed drawings of a more elaborate home-made cement-testing machine, see *Engineering News*, vol. xv. p. 310, or Proceedings Engineers' Club of Philadelphia.

88. Mixing the Mortar.—It is customary to test some briquettes made wholly of cement and also some composed of part sand and part cement, the latter being a check upon the tests for fineness. For example, finely ground cements, as compared with coarsely ground, will give lower results tested neat, and higher ones with liberal proportions of sand. On the other hand, unevenly ground cement—that is, part very fine, and part very coarse—shows, in comparison with other grades, a high strength when tested neat, and gives very low results when mixed with sand. In the first case the coarse particles act like sand, and in the second there are not enough fine particles to envelop the coarse particles of cement and sand.

"For tests of mortar composed in part of sand, the sand should be sharp, well washed and dried, rejecting all that will not pass the sieve of 400 meshes to the square inch and that will pass a sieve of 900 meshes to the square inch,—the wire gauge of the former sieve to be No. 24, and of the latter No. 28."

In comparing different cements great care should be taken to have the sand of the same quality in each case (§ 116), since the variation of strength of a cement mixed with different qualities of sand is often more than the difference in strength between different brands.

"The proportion of sand, for the mortar of each briquette, should be carefully determined by weight, and thoroughly and intimately mixed with the cement in a dry state before water is added; and, so

far as possible, all the water that is necessary to produce the desired consistency of the resulting mortar should be added at once, and thereafter the manipulation by the spatula or trowel should be rapid and thorough. Care should be taken to introduce the mass into the moulds and complete the moulding process before incipient setting begins." * For additional instructions as to mixing the mortar, see §§ 96 and 99.

The moulds, while being charged and manipulated, should be laid on glass, slate, or some other non-absorbent material. The briquette should be removed from the moulds as soon as it is hard enough to stand it. The briquettes should be immersed in water at the end of 24 hours, and should be tested as soon as taken from the water. If they are exposed to the air, the water may be carried away by evaporization and leave the mortar a pulverulent mass. Also, since the mortar does not ordinarily set as rapidly under water as in the air (owing to the difference in temperature), it is necessary, for accurate work, to note the time of immersion, and also to break the briquette as soon as it is taken from the water. Notice that, ordinarily, cement attains a greater strength when allowed to set under water, but attains it more slowly.

89. Age of Mortar. It is customary to break part of the briquettes at the end of 7 days and the remainder at the end of 28 days. As it is sometimes impracticable to wait 28 days, tests are often made at the end of 1 and 7 days respectively. The ultimate strength of the cement is judged by the increase in strength between the two dates. A minimum strength for the two dates is usually specified.

90. Data on Strength. The report of the Committee of the American Society of Civil Engineers, on Uniform Tests of Cements (Transactions of that Society, vol. xiv. pp. 478–79), gives the results in the following table as "the average minimum and maximum tensile strength per square inch which some good cements have attained" when tested under the conditions specified above.

The quantity in the "Min." columns of the following table is the average strength, for the time specified, of the weaker brands of each of the two classes of cement; and similarly the quantity in the "Max." columns is the average strength for the stronger brands. In making tests with any sample of cement, the results are liable to considerable variation, depending upon the details of the manipula-

* Report of the Committee of the American Society of Civil Engineers.

TABLE 10.

TENSILE STRENGTH OF CEMENT MORTARS.

AGE OF MORTAR WHEN TESTED.	AVERAGE TENSILE STRENGTH IN POUNDS PER SQUARE INCH.			
	Portland.		Rosendale.	
CLEAR CEMENT.	Min.	Max.	Min.	Max.
1 day—1 hour, or until set, in air, the remainder of the time in water........................	100	140	40	80
1 week—1 day in air, the remainder of the time in water..................................	250	550	60	100
4 weeks—1 day in air, the remainder of the time in water..................................	350	700	100	150
1 year—1 day in air, the remainder of the time in water..................................	450	800	300	400
1 PART CEMENT TO 1 PART SAND.				
1 week—1 day in air, the remainder of the time in water..................................	......	......	30	50
4 weeks—1 day in air, the remainder of the time in water..................................	......	......	50	80
1 year—1 day in air, the remainder of the time in water..................................	......	......	200	300
1 PART CEMENT TO 3 PARTS SAND.				
1 week—1 day in air, the remainder of the time in water..................................	80	125	......	......
4 weeks—1 day in air, the remainder of the time in water..................................	100	200	......	......
1 year—1 day in air, the remainder of the time in water..................................	200	350	......	......

tion of the mortar and the machine.* Even if the same kind and quantity of cement, sand, and water are used, there is liability to an extreme variation of 20 to 25 per cent. in the average result. Hence, to properly test any sample of cement, it is essential that several trials should be made. This is especially important when one invoice is to be compared with another or with a standard, since the details of the experiments are apt to vary from day to day, although the experimenter may be unconscious of it. In making tests of the strength of cements, it should be borne in mind that the strength

* See foot-note, page 63.

depends upon the activity, soundness, and fineness of the cement, and also upon the amount and kind of aggregate, upon the amount and temperature of the water, and upon the details of the manipulation in making and testing. Each item affects the strength to an appreciable degree.

For additional data on the strength of mortars composed of different proportions of cement and sand, see § 134.

91. Weight and Cost of Cement. Cement is generally sold by the barrel. A barrel of Portland cement weighs 400 pounds gross, or about 375 net. Ulster Co. Rosendale cement weighs 300 pounds per barrel net; Akron, Milwaukee, Utica, and Louisville Rosendales weigh 265 pounds per barrel net. Cement is also occasionally shipped in bags and in bulk, in which cases it is sold by weight.

The average price of cement of the Rosendale type at Chicago during 1887 was $1.00 to $1.25 per barrel; Portland averaged from $3.00 to $3.25 per barrel.

Art. 5. Specifications for Cement.

92. Cement is so variable in quality and intrinsic value that no considerable quantity should be accepted without testing it to see that it conforms to a specified standard. A careful study of the preceding section will enable any one to prepare such specifications as will suit the special requirements, and also give the instructions necessary for applying the tests. In many of the European countries a uniform system has been devised and put in force by the respective governments. A few of these specifications will be given to serve as guides in preparing others.

Abstracts from German Specifications for Standard Portland Cement.*

93. "**Constancy of Volume.** Portland cement shall be of constant volume. As a preliminary test, admitting of forming a rapid opinion, the heat test [described in the next paragraph] is recommended. The decisive test shall be that a paste of neat cement, made on a glass plate, protected against drying, and placed

* *Engineering News*, vol. xvi. pp. 316–17. Translated for the Laboratory of the Department of Civil Engineering of Cornell University, Ithaca, N. Y.

under water, after twenty-four hours shall not show any blowing cracks or change of shape.

"For making the heat test, a stiff paste of neat cement and water is made, and from this, cakes 8 centimetres (3.15 inches) to 10 centimetres (3.94 inches) in diameter and 1 centimetre (0.394 inch) thick, are formed on a smooth impermeable plate covered with blotting-paper. Two of these cakes, which are to be protected against drying in order to prevent drying cracks, are placed, after the lapse of twenty-four hours, or at least only after they have set, with their smooth surfaces on a metal plate and exposed, for at least an hour, to a temperature of from 110° C. to 120° C. (230° to 248° F.) until no more water escapes. If, after this treatment, the cakes show no edge cracks, the cement is to be considered, in general, of constant volume. If such cracks do appear, the cement is not to be condemned, but the results of the decisive test with the cakes hardening on glass plates under water must be waited for. It must, however, be noticed that the heat test does not admit of a final conclusion as to the constancy of volume of those cements which contain more than 3 per cent. of calcium sulphate (gypsum) or other sulphur combinations.

"For making the final test, the cake made for the purpose of determining the time of setting, for slow-setting cements, is placed under water after the lapse of twenty-four hours—or, at all events, not until after it is set. For quick-setting cements, this can be done after a shorter period. The cakes, especially those of slow-setting cements, must be protected against draughts and sunshine until their final setting. This is best accomplished by keeping them in a covered box lined with zinc, or under wet cloths. In this manner the formation of heat-cracks is avoided, which are generally formed in the center of the cake, and may be taken by an inexperienced person for cracks formed by blowing.

94. "**Fineness of Grinding.** Portland cement shall be so finely ground that a batch of the same shall not leave a residue of more than 10 per cent. upon a sieve of 900 meshes per square centimetre (5,806 meshes per square inch). The thickness of the wire of the sieve shall equal half the space between the wires. For each test batch, 100 grammes (3½ oz.) of cement shall be used.

95. "**Time of Setting.** To determine the time of setting, a slow-setting neat cement shall be mixed three minutes, and a quick-

setting neat cement one minute, with water to a stiff paste. A cake about 1.5 centimetres (0.59 inch) thick, with thin edges, shall be formed of this paste on a plate of glass. The consistency of the cement paste for this cake shall be such that, when wrought with a trowel on the plate, the paste will only begin to run towards the edge of the same after the paste has been repeatedly jarred. As a rule, 27 to 30 per cent. of water will suffice to give the necessary consistency to the paste. As soon as the cake is sufficiently hardened, so that it will resist a slight pressure of the finger-nail, the cement is to be considered as having set.

"For the exact determination of the time of setting, and for determining the beginning of the time of setting,—which latter is of importance in the case of quick-setting cements, since they must be worked up before they begin to set,—a standard needle 300 grammes (10 oz.) in weight, and 1 square millimetre (0.0006 square inch) in cross section is used. A metal ring 4 centimetres (1.575 inches) in height and 8 centimetres (3.15 inches) clear diameter (inside) is placed on a glass plate, filled with cement paste of the above consistency, and brought under the needle. The moment at which the needle is no longer capable of completely penetrating the cement cakes is considered the beginning of the time of setting. The time elapsing between this and the moment when the standard needle no longer leaves an appreciable impression on the hardened cake is considered the time of setting."

"Cements which do not set in less than two hours are to be considered slow-setting cements.

96. "**Tests of Strength.** The cohesive power of Portland cement shall be determined by the testing of a mixture of cement and sand. At the same time a determination of the strength of the neat cement is to be recommended. The tests shall be both tensile and compressive, with test pieces of the same form and section. For each test, in order to obtain correct average results, at least six test pieces are to be made, and the mean of the four best results is to be considered the final strength. The tensile test pieces may be made either by hand or by machinery; but the compressive test pieces must be made by machinery.

Sand. "In order to obtain concordant results in the tests, sand of uniform size of grain and uniform quality must be used. This standard sand is obtained by washing and drying the purest quartz

sand obtainable, sifting the same through a sieve with 60 meshes per square centimetre (387 per square inch), thereby separating the coarsest particles, and by removing from the sand so obtained, by means of a sieve of 120 meshes per square centimetre (774 per square inch), the finest particles. The diameter for the wires of the sieves shall be 0.38 millimetres and 0.32 millimetres (0.015 inch and 0.013 inch) respectively.

Hand-Mixing. "On a metal or thick glass plate five sheets of blotting-paper soaked in water are laid, and on these are placed five moulds wetted with water. 250 grammes (8.75 oz.) of cement and 750 grammes (26.25 oz.) of standard sand are weighed, and thoroughly mixed dry in a vessel. Then 100 cubic centimetres (100 grammes or 35 oz.) of fresh water are added, and the whole mass thoroughly mixed for five minutes. With the mortar so obtained, the moulds are at once filled, with one filling, so high as to be rounded on top, the mortar being well pressed in. By means of an iron trowel 5 to 8 centimetres (1.96 inches to 3.14 inches) wide, 35 centimetres (13.79 inches) long, and weighing about 250 grammes (8.75 oz.), the projecting mortar is pounded, first gently and from the side, then harder into the moulds, until the mortar grows elastic and water flushes to the surface. A pounding of at least one minute is absolutely essential. An additional filling and pounding in of the mortar is not admissible, since the test pieces of the same cement should have the same densities at the different testing stations. The mass projecting over the mould is now cut off with a knife, and the surface smoothed. The mould is carefully taken off and the test piece placed in a box lined with zinc, which is to be provided with a cover, to prevent a non-uniform drying of the test pieces at different temperatures. Twenty-four hours after being made, the test pieces are placed under water, and care must be taken that they remain under water during the whole period of hardening.

Machine-Mixing. "After the mould has been clamped on the bed-plate of the pounding machine, for each test 180 grammes (6.3 oz.) of the mortar, made as above, is placed in the mould, and the iron follower is set in. By means of Boehme's hammer apparatus,* with a hammer weighing 2 kilogrammes (4.4 pounds) 150 blows are struck on the follower.

* For illustrated description, see *Engineering News*, vol. xvii. p. 200.

97. "Good quick-setting cement, in the proportion of 3 parts by weight of standard sand to 1 part of cement, when tested after 28 days' hardening (1 in air and 27 in water), shall have a minimum tensile strength of at least 16 kilogrammes per square centimetre (227 pounds per square inch). The compressive strength shall be at least 10 times the tensile strength.

"For slow-setting cements, the strength after 20 days is less in general than the one above specified; therefore, in giving the results of tests, the time of setting shall also be given."

ABSTRACTS FROM FRENCH SPECIFICATIONS FOR PORTLAND CEMENT.*

98. "**CHEMICAL ANALYSIS.** The cement must not contain more than 1 per cent. of sulphuric acid or sulphides in determinable proportion.† Cements containing more than 4 per cent. of ferric oxide, or in which the ratio of the combined silica and alumina to the lime is less than 0.44, are to be regarded as doubtful.

99. "**MIXING THE MORTAR.** In mixing the mortar for testing, sea water is specified, and both air and water are to be maintained at a temperature of 15° to 18° C. (59° to 64.4° F.) during the continuance of the experiments. The quantity of water is ascertained by a preliminary experiment, and the four following tests are given to serve as an indication whether the proportion of water added is correct:

"1. The consistence of the mortar should not change if it be gauged for an additional period of three minutes after the initial five minutes.

"2. A small quantity of the mortar dropped from the trowel upon the marble slab from a height of about 0.50 metres (1.64 ft.) should leave the trowel clean, and retain its form approximately without cracking.

"3. A small quantity of the mortar worked gently in the hands should be easily moulded into a ball, on the surface of which water should appear. When this ball is dropped from a height of 0.50 metres (1.64 ft.), it should retain a rounded shape without cracking.

* From Abstracts of Inst. of C. E.

† The percentage of sulphides in the cement is said to be an indication of adulteration with blast-furnace slag.

"4. If a slightly smaller quantity of water be used, the mortar should be crumbly, and crack when dropped upon the slab. On the other hand, the addition of a further quantity of water—1 to 2 per cent. of the weight of the cement—would soften the mortar, rendering it more adhesive, and preventing it from retaining its form when allowed to fall upon the slab. It is recommended to commence with a rather smaller quantity of water than may be ultimately required, and then to make fresh mixings with a slight additional quantity of water.

"The mortar is to be mixed with a trowel for five minutes upon a marble slab."

100. STRENGTH. The form of briquette and method of moulding are the same as required by the German specifications (see § 96); the breaking section is 5 square centimetres (0.775 square inch).

"Six briquettes are broken after an interval of 7 days, six after 28 days, and the remaining six after 84 days. The mean of the three highest figures of each series of tests is taken as the tensile strength of the cement under examination. The minimum strength specified for the neat cement in 7 days is 20 kilogrammes per square centimetre (284.5 lbs. per sq. in.); in 28 days, 35 kilogrammes per square centimetre (497.8 lbs. per sq. in.); and at least 45 kilogrammes per square centimetre (640 lbs. per sq. in.) in 84 days. If, however, the strength in 28 days is not more than 5 kilogrammes per square centimetre (71.12 lbs. per sq. in.) in excess of that at 7 days, then it must be at least 55 kilogrammes per square centimetre (782.27 lbs. per sq. in.) in 28 days, and in any case where this strength is not attained at 28 days it must be exceeded in 84.

101. "Tests of cement mixed with sand are also specified. The standard sand is produced by crushing quartzite obtained from quarries near Cherbourg, and sifting it through sieves of 64 and 144 meshes per square centimetre (413 and 929 meshes per square inch).* That which remains between these two sieves is washed and dried, and constitutes the standard sand. 375 grammes (13.28 oz.) of this sand is mixed with 125 grammes (4.41 oz.) of cement, and water is added in the proportion of 12 parts by weight to 100 parts of sand and cement combined. The sand and cement are first carefully

* "The size of mesh of the sieve is not clearly specified, the thickness of the wire not being stated."

mixed in a basin or capsule, then the whole of the sea water is added at once, and the mixture stirred with a spatula for five minutes.

"At the expiration of 7 days the strength of the sand-cement briquettes should be at least 8 kilogrammes per square centimetre (113.78 lbs. per sq. in.), and in 28 days 15 kilogrammes per square centimetre (213.35 lbs. per sq. in.). In 28 days the strength should exceed that at 7 days by 2 kilogrammes per square centimetre (28.45 lbs. per sq. in.). In 84 days the strength must be greater than at 28 days, and at least 18 kilogrammes per square centimetre (256 lbs. per sq. in.). The 84-day tests are only considered indispensable for those cements which may have stood the two previous tests; but if, while the cement is in store, the 84-day tests should be unsatisfactory, it may be rejected."

102. Fineness of Grinding. "The degree of fineness to which the cement must be ground is not specified, it being considered that very fine grinding increases the strength chiefly during the duration of the tests, and that the subsequent increase of strength is less with fine than with coarse cement."

103. Time of Setting. Essentially the same as the German specifications; see § 95.

"Any cement commencing to set in less than 30 minutes, or failing to commence to set within 3 hours, is to be rejected; and the final set must have taken place within 12 hours. In each case the time is reckoned from the moment the water is poured upon the cement."

Austrian Specifications for Fineness and Strength of Cement.*

104. "**For Portland.** *Fineness*, not more than 20 per cent. to be left on sieve of 5,806 meshes per square inch. *Tensile strength* (1 part cement and 3 parts sand), 1 day in air and 6 in water, 113.78 lbs. per sq. in.; 1 day in air and 27 in water, 170.68 lbs. per sq. in.

105. "**For Roman.**† *Fineness*, same as for Portland. *Tensile strength* (1 part cement and 3 parts sand): For quick-setting cements (taking 15 minutes, or less, to set), 1 day in air and 6 days

* Report of Committee on Uniform Tests of Cement, in Trans. Am. Soc. of C. E., vol. xiv. p. 480.

† Essentially the same cement as Rosendale; see § 72.

in water, 23 lbs. per sq. in.; 1 day in air and 27 in water, 56.9 lbs. per sq. in. For slow-setting cements (taking more than 15 minutes to set), 1 day in air and 6 days in water, 42.6 lbs. per sq. in.; 1 day in air and 27 in water, 85.3 lbs. per sq. in."

English Specifications for Portland Cement.

The following is a summary of the specifications used by Mr. Henry Faija, an accepted English authority.*

106. "**Fineness** to be such that the cement will all pass through a sieve having 625 holes (25^2) to the square inch, and leave only 10 per cent. residue when sifted through a sieve having 2,500 holes (50^2) to the square inch.

107. "**Expansion or Contraction.** A pat made and submitted to moist heat and warm water at a temperature of about 100° F., shall show no sign of blowing in twenty-four hours.

108. "**Tensile Strength.** Briquettes of slow-setting Portland, which have been gauged, treated, and tested in the prescribed manner, to carry an average tensile strain, without fracture, of at least 176 lbs. per sq. in. at the expiration of 3 days from gauging; and those tested at the expiration of 7 days, to show an increase of at least 50 per cent. over the strength of those at 3 days, but to carry a minimum of 350 lbs. per sq. in.

"For quick-setting Portland, at least 176 lbs. per sq. in. at 3 days, and an increase at 7 days of 20 to 25 per cent., but a minimum of 400 lbs. per sq. in. Very high tensile strengths at early dates generally indicate a cement verging on an unsound one."

Tests Recommended by the Committee of the American Society of Civil Engineers.†

109. "It is recommended that tests for hydraulic cement be confined to methods for determining fineness, liability to checking or cracking, and tensile strength; and for the latter, for tests of 7 days and upward, that a mixture of 1 part of cement to 1 part of sand for natural cements, and 3 parts of sand for Portland cements,

* As given by Mr. Faija in Trans. Am. Soc. of C. E., vol. xvii. p. 225.
† Trans. Am. Soc. of C. E., vol. xiv. pp. 478–85.

be used, in addition to trials of neat cement. The quantities used in the mixture should be determined by weight." For the form of test specimen, method of mixing, etc., see § 87 and § 88.

110. "SAMPLING. There is no uniformity of practice among engineers as to the sampling of the cement to be tested, some testing every tenth barrel, others every fifth, and others still every barrel delivered. Usually, where cement has a good reputation, and is used in large masses, such as concrete in heavy foundations or in the backing or hearting of thick walls, the testing of every fifth barrel seems to be sufficient; but in very important work, where the strength of each barrel may in a great measure determine the strength of that portion of the work where it is used, or in the thin walls of sewers, etc., etc., every barrel should be tested, one briquette being made from it.

"In selecting cement for experimental purposes, take the samples from the interior of the original packages at sufficient depth to insure a fair exponent of the quality, and store the same in tightly-closed receptacles impervious to light or dampness until required for manipulation, when each sample of cement should be so thoroughly mixed, by sifting or otherwise, that it shall be uniform in character throughout its mass.

111. "CHECKING OR CRACKING. The test for checking or cracking is an important one, and, though simple, should never be omitted. It is as follows:

"Make two cakes of neat cement 2 or 3 inches in diameter, about $\frac{1}{2}$ inch thick, with thin edges. Note the time in minutes that these cakes, when mixed with water to the consistency of a stiff plastic mortar, take to set hard enough to stand the wire test recommended by Gen. Gillmore,—$\frac{1}{12}$-inch diameter wire loaded with one fourth of a pound, and $\frac{1}{24}$-inch loaded with 1 pound (see § 82). One of these cakes, when hard enough, should be put in water and examined from day to day to see if it becomes contorted or if cracks show themselves at the edges, such contortions or cracks indicating that the cement is unfit for use at that time. In some cases the tendency to crack, if caused by the presence of too much unslaked lime, will disappear with age. The remaining cake should be kept in the air and its color observed, which, for a good cement, should be uniform throughout (yellowish blotches indicating a poor quality), the Portland cements being of a bluish-

gray, and the natural cements being light or dark according to the character of the rock of which they are made. The color of the cements when left in the air indicates the quality much better than when they are put in water.

112. "**Fineness.** The strength of a cement depends greatly upon the fineness to which it is ground, especially when mixed with a large dose of sand. It is therefore recommended that the tests be made with cement that has passed through a No. 100 sieve (10,000 meshes to the square inch), made of No. 40 wire (Stubs's wire gauge). The results thus obtained will indicate the grade which the cement can attain, under the condition that it is finely ground, but it does not show whether or not a given cement offered for sale shall be accepted and used. The determination of this question requires that the tests should also be applied to the cement as found in the market. Its quality may be so high that it will stand the tests, even if very coarse and granular; and, on the other hand, it may be so low that no amount of pulverization can redeem it. In other words, fineness is no sure indication of the value of a cement, although all cements are improved by fine grinding. Cement of the better grades is now usually ground so fine that only from 5 to 10 per cent. is rejected by a sieve of 2,500 meshes per square inch, and it has been made so fine that only from 3 to 10 per cent. is rejected by a sieve of 32,000 meshes per square inch. The finer the cement, if otherwise good, the larger dose of sand it will take, and the greater its value.

113. "**Sieves.** For ascertaining the fineness of *cement*, it will be convenient to use three sieves; viz., No. 50 (2,500 meshes to the square inch), wire to be of No. 35 Stubs's wire gauge; No. 74 (5,476 meshes to the square inch), wire to be of No. 37 Stubs's wire gauge; No. 100 (10,000 meshes to the square inch), wire to be of No. 40 Stubs's wire gauge. The object is to determine by weight the percentage of each sample that is rejected by these sieves, with a view not only of furnishing the means of comparison between tests made of different cements by different observers, but indicating to the manufacturer the capacity of his cement for improvement in a direction always and easily within his reach.

"For ascertaining the fineness of *sand*, two sieves are recommended; viz., No. 20 (400 meshes to the square inch), wire to be

of No. 28 Stubs's wire gauge; No. 30 (900 meshes to the square inch), wire to be of No. 31 Stubs's wire gauge.

"These sieves can be furnished in sets as follows, an arrangement having been made with a manufacturer* of such articles, by which he agrees to furnish them of the best quality of brass-wire cloth, set in metal frames, the cloth to be as true to count as it is possible to make it, and the wire to be of the required gauge. Each set will be inclosed in a box, the sieves being nested.

Set A, three cement sieves, to cost $4.80:	
No. 100	7 inches diameter.
No. 74	6½ " "
No. 50	6 " "
Set B, two sand sieves, to cost $4.00:	
No. 80	8 inches diameter.
No. 20	7½ " "

114. "TENSILE STRENGTH. The tests should be applied to the cements as offered for sale.† If satisfactory results are obtained with a full dose of sand, the trials need go no further. If not, the coarser particles should first be excluded by using a No. 100 sieve, in order to determine approximately the grade the cement would take if ground fine, for fineness is always attainable, while inherent merit may not be.

"*Standard Sand.* The question of a standard sand seems one of great importance, for it has been found that sands looking alike and sifted through the same sieves give results varying within rather wide limits.

"The material that seems likely to give the best results is the crushed quartz used in the manufacture of sand-paper. It is a commercial product, made in large quantities and of standard grades, and can be furnished of a fairly uniform quality. It is clean and sharp, and although the present price is somewhat excessive (3 cents per pound), it is believed that it can be furnished in quantity for about $5.00 per barrel of 300 lbs. As it would be used for tests only, for purposes of comparison with the local sands, and with tests of different cements, not much of it would be required. The price of the German standard sand is about $1.25

* "Williams's Globe Wire Works, 85 Fulton Street, New York City."

† For the table of values recommended by the committee, see page 66.

per 112 pounds; but the article, being washed river sand, is probably inferior to crushed quartz. Crushed granite could be furnished at a somewhat less rate than quartz, and crushed trap for about the same as granite; but no satisfactory estimate has been obtained for either of these.

"The use of crushed quartz is recommended by your Committee, the degree of fineness to be such that it will all pass a No. 20 sieve and be caught on a No. 30 sieve. Of the regular grade of crushed quartz—No. 3,—from 15 to 37 per cent. passes a No. 30 sieve, and none of it passes a No. 50 sieve. As at present furnished, it would need resifting to bring it to the standard size; but if there were sufficient demand to warrant it, it could undoubtedly be furnished of the size of grain required at little, if any, extra expense."

PART II.

METHODS OF PREPARING AND USING THE MATERIALS.

CHAPTER IV.

MORTAR, CONCRETE, AND ARTIFICIAL STONE.

ART. 1. MORTAR.

115. Mortar is a mixture of the paste of cement or lime with sand. The paste may be made before adding the sand, or the materials may be incorporated dry and afterwards tempered to a plastic condition with water. In common mortar, the cementing substance is ordinary lime; in hydraulic mortar, it is hydraulic cement.

116. SAND. Sand is mixed with lime or cement to reduce the cost of the mortar; and is added to lime also to prevent the cracking which would occur if lime were used alone. Any material may be used to dilute the mortar, provided it has no effect upon the durability of the cementing material and is not itself liable to decay. Burnt clay, powdered brick, slag, or coal cinders may be used. Of course the strength of the mortar decreases with the amount of dilution.

117. Requisites for Good Sand. It is usually specified that sand for mortar should be clean, and sharp, and free from pebbles.

Although it is customary to require that only clean sand shall be used in mixing mortar, a small amount of finely powdered, inert diluting material, as clay for example, will not materially decrease the strength of the mortar. Clay, when dissolved or finely pulverized, consists of an almost impalpable powder; and when mixed with the sand, its particles occupy the interstices between the par-

ticles of cement and sand, and are also completely enveloped by the cementing paste. Clay, dissolved or finely pulverized, mixed with cement up to the proportion of 1 to 1, appears to affect the strength essentially the same as an equal quantity of sand;* and the mortar is much more dense, plastic, and water-tight, and is occasionally convenient for plastering surfaces and stopping leaky joints. Such mortar is not affected by the presence of water.

The cleanness of sand may be tested by rubbing a little of the dry sand in the palm of the hand, and after throwing it out noticing the amount of dust left on the hand. The cleanness of sand may also be judged by pressing it together between the fingers while it is damp; if the sand is clean, it will not stick together, but immediately fall apart when the pressure is removed. As a rule, reasonably clean sand can be had without any extra trouble or expense.

Sharp sand, *i.e.*, sand with angular grains, is much better than that with rounded grains, although it is often difficult to obtain. The sharpness of sand can be determined approximately by rubbing a few grains in the hand, or by crushing it near the ear and noting if a grating sound is produced; but an examination through a small lens is better. The requirement that "the sand shall be sharp" is practically a dead letter in most specifications.

Sand should be screened to remove the pebbles, the fineness of the screen depending upon the kind of work in which the mortar is to be used. Every particle of the sand or "aggregate" should be completely covered with the cement or "matrix;" and since when the grains in a given volume are small the magnitude of the total surface to be covered is greater than when the grains are large, it follows that fine sand requires a larger proportion of cement than coarse sand. Any specification or plan contemplating the use of coarse sand must, therefore, be altered if fine sand alone is used, else the quality of the mortar will be impaired. The best sand is that in which the grains are of different sizes. The more uneven the sizes, the smaller the voids, and hence less cement is required. The voids of ordinary sand average from 0.3 to 0.5 of the volume.

118. Cost and Weight of Sand. The price of reasonably good sand varies from 40 cents to $1.60 per yard, according to locality.

* Trans. Am. Soc. of C. E., vol. xiv. p. 164.

At Chicago, in 1887, sand cost on the average 40 cents per cubic yard, delivered at the work.

Sand is sometimes sold by the ton. It weighs, when dry, from 80 to 115 lbs. per cu. ft., or about 1 to 1¼ tons per cubic yard.

119. Common Lime Mortar. Mortar made of the paste of common or fat lime is extensively used on account of (1) its intrinsic cheapness, (2) its great economic advantage owing to its great increase of volume in slaking, and (3) the simplicity attending the mixing of the mortar. On account of the augmentation of volume, the paste of fat lime shrinks in hardening, to such an extent that it cannot be employed as mortar without a large dose of sand.

As a paste of common lime sets or hardens very slowly, even in the open air, unless it be subdivided into small particles or thin films, it is important that the volume of lime paste in common mortar should be but slightly in excess of what is sufficient to coat all the grains of sand and to fill the voids between them. If this limit be exceeded, the strength of the mortar will be impaired. With most sands the proper proportions will be from 2.5 to 3 volumes of sand to 1 volume of lime paste. Generally, if either less or more sand than this be used, the mortar will be injured,—in the former case from excess of lime paste, and in the latter from porosity. Notice that the volume of the resulting mortar is about equal to the volume of the sand alone.

120. The ordinary method of slaking lime consists in placing the lumps in a layer 6 or 8 inches deep in either a water-tight box, or a basin formed in the sand to be used in mixing the mortar, and pouring upon the lumps a quantity of water 2½ to 3 times the volume of the lime.

This process is liable to great abuse at the hands of the workmen. They are apt either to use too much water, which reduces the slaked lime to a semi-fluid condition and thereby injures its binding qualities; or, not having used enough water in the first place, to seek to remedy the error by adding more after the slaking has well progressed and a portion of the lime is already reduced to powder, thus suddenly depressing the temperature and chilling the lime, which renders it granular and lumpy. It is also very important that the lime should not be stirred while slaking. The essential point is to secure the reduction of all the lumps. Covering the bed of lime with a tarpaulin or with a layer of sand retains the

heat and accelerates the slaking. All the lime necessary for any required quantity of mortar should be slaked at least one day before it is incorporated with the sand.

After the lime is slaked the sand is spread evenly over the paste, and the ingredients are thoroughly mixed with a shovel or hoe, a little water being added occasionally if the mortar is too stiff.

One barrel (230 lbs.) of lime will make about 8 cubic feet of stiff paste.

121. The common mortar of quick-lime and sand is not fit for thick walls, because it depends upon the slow action of the atmosphere for hardening it; and, being excluded from the air by the surrounding masonry, the mortar in the interior of the mass hardens only after the lapse of years, or perhaps never.* The mortar of cement, if of good quality, sets immediately; and, as far as is known, continues forever to harden without contact with the air. Cement mortar is the only material whose strength increases with age. Owing to its not setting when excluded from the air, common lime mortar should never be used for masonry construction under water, or in soil that is constantly wet; and, owing to its weakness, it is unsuitable for structures requiring great strength, or subject to shock. Its use in engineering masonry has been abandoned on all first-class railroads. Cement is so cheap, that it could profitably be substituted for lime in the mortar for ordinary masonry.

122. Hydraulic Lime Mortar. With mortars of hydraulic lime the volume of sand should not be less than 1.8 times that of the lime paste, in order to secure the best results regardless of cost. The usual proportions are, however, for ordinary work, the same as in common mortars, care being taken to incorporate sufficient paste to coat all the grains of sand and to fill up the voids between them.

123. Hydraulic Cement Mortar. A paste of good hydraulic cement hardens simultaneously and uniformly throughout the mass, and its strength is impaired by any addition of sand. The relative quantities of sand and cement depend somewhat upon the kind of work and upon the conditions of the ingredients when measured. For ordinary use, it is customary to add as much sand as is possible without making the mortar porous. The proportions may vary

* Lime mortar taken from the walls of ancient buildings has been found to be only 50 to 80 per cent. saturated with carbonic acid after nearly 2,000 years of exposure. Lime mortar 2,000 years old has been found in subterranean vaults, in exactly the condition, except for a thin crust on top, of freshly mixed mortar.

from 1 part of cement and 2 parts of sand to 1 part of the former and 4 of the latter. See § 134.

When the mortar is required in small quantities, as for use in ordinary masonry, it is mixed about as follows: About half the sand to be used in a batch of mortar is spread evenly over the bed of the mortar-box; and then the dry cement is spread evenly over the sand; and finally the remainder of the sand is spread on top. The sand and cement are then mixed with a hoe or by turning and re-turning with a shovel.* It is very important that the sand and cement be thoroughly mixed.

The dry mixture is then shoveled to one end of the box, and water is poured into the other end. Cements vary greatly in their capacity for water (see §§ 82 and 99), freshly-ground cements requiring more than those that have become stale. An excess of water is, however, better than a deficiency, particularly when a very energetic cement is used, as the capacity of this substance for solidifying water is great. The sand and cement are then drawn down with a hoe, small quantities at a time, and mixed with the water until enough has been added to make a good stiff mortar. This should be vigorously worked with a hoe for several minutes, to insure a good mixture. The mortar should then leave the hoe clean when drawn out of it, and very little should stick to the steel.

124. When mortar is required in considerable quantities, as in mixing concrete, it is mixed by machinery. See § 152.

125. Hydraulic cements set better and attain a greater strength under water than in the open air; in the latter, owing to the evaporation of the water, the mortar is liable to dry instead of setting. This difference is very marked in hot, dry weather. If cement mortar is to be exposed to the air, it should be shielded from the direct rays of the sun, and kept moist by sprinkling or otherwise.

126. Rosendale vs. Portland Cement. It is sometimes a question whether Portland or Rosendale cement should be preferred. Generally this question should be decided on economical grounds, which makes it a question of relative strengths (see § 134) and relative prices (see § 138); but frequently it is determined by other considerations. Thus if great ultimate strength is required, then Portland cement must be used; but if a quick-setting mortar is desired, then Rosendale cement must be employed.

* See the last paragraph of § 260.

Unless a quick-setting mortar is required, there is a decided advantage in using Portland; for as it hardens more slowly, it is not so liable to set before reaching its place in the wall. This is an important item, since with quick-setting cement any slight delay might necessitate the throwing away of a boxful of mortar or the removal of a stone to scrape out the partially-set mortar.

It is sometimes very desirable to have a cement which will set more quickly than Portland, and finally attain a greater strength than Rosendale. Under such circumstances a mixture of Portland and Rosendale can be used. "Such mortar sets about as quickly as if made with Rosendale cement alone, and acquires great subsequent strength, due to the Portland cement contained in it. This was proven by many experimental tests."* The strength of the mixed mortar is almost exactly a mean between that of the two mortars separate.

127. Lime with Cement. The advantages of a slow-setting mortar can be obtained by mixing common lime with a rich Rosendale mortar. *The lime should be reduced to a paste before being added to the cement.* The addition of the lime gives the double advantage of a rather slow-setting mortar and a cheap one, but decreases the strength of the mortar. No experiments seem to have been made to determine the activity of a mixture of cement and lime; but from practical experience it is well known that the addition of lime somewhat retards the setting of cement mortar. The extent to which the induration of different cements is affected by the addition of lime seems to vary directly with their activity.

It has long been an American practice to reinforce lime mortar by the addition of hydraulic cement. The mortar for the "ordinary brick-work" of the United States public buildings is composed of "one fourth cement, one half sand, and one fourth lime." Of course a cement mortar is better, but it costs more.

For the strength of a mortar composed of lime and cement, see § 135.

128. GROUT. This is merely a thin or liquid mortar of lime or cement. The interior of a wall is sometimes laid up dry, and the grout, which is poured on top of the wall, is expected to find its way downwards and fill all voids, thus making a solid mass of the wall.

* Trans. Am. Soc. of C. E., vol. xiv. p. 163.

Grout should never be used when it can be avoided. If made thin, the water only slowly dries out of the wall; and if made thick, the grout fills only the upper portion of the wall. To get the greatest strength, the mortar should have only enough water to make a stiff paste—the less water the better. If the mortar is stiff, the brick or stone should be dampened before laying; else the brick will absorb the water from the mortar before it can set, and thus destroy the adhesion of the mortar.

129. Data for Estimates. The following data will be found useful in estimating the amounts of the different ingredients necessary to produce any required quantity of mortar:—

One barrel of lime (230 lbs.) will make about 2¼ barrels (0.3 cu. yd.) of stiff lime paste. One barrel of lime paste and three barrels of sand will make about three barrels (0.4 cu. yds.) of good lime mortar. One barrel of unslaked lime will make about 6.75 barrels (0.95 cu. yd.) of 1 to 3 mortar.

A barrel of Portland cement weighs 400 pounds gross, or about 375 pounds net. Hudson River Rosendale weighs 300 pounds net per barrel. Western Rosendale weighs 265 pounds net per barrel.

A barrel of Rosendale, as packed at the manufactories on the Hudson, will measure from 1.25 to 1.40 barrels, if measured loose. A barrel of Western Rosendale will make about 1.1 barrels, if measured loose. A commercial barrel of Portland will make about 1.2 barrels, if measured loose.

One cubic foot of dry cement (shaken down, but not compressed) mixed with 0.33 cubic feet of water will give 0.63 cubic feet of stiff paste. One barrel (300 pounds) of finely-ground Rosendale cement will make from 3.70 to 3.75 cubic feet of stiff paste; or 79 to 83 pounds of cement powder will make about one cubic foot of stiff paste. Volume for volume, Portland will make about the same amount of paste as Rosendale; or 100 pounds of Portland will make a cubic foot of stiff mortar.

130. The proportions of sand and cement are generally measured by volumes. In mixing mortars for experimental purposes both the cement and the sand are measured loose; but in actual work the cement is usually delivered in barrels, and consequently the most convenient unit for the cement is a commercial barrel, while it is most convenient to measure the sand loose. Cement is some-

times shipped in bags, but as an integral number of bags, usually three or four, are equal in weight to a barrel, the barrel of packed cement and the barrel of loose sand are still the most convenient units. Occasionally cement is shipped in bulk, in which case the quantity of cement required may be determined by weighing or by measuring by volumes loose. If the cement is weighed, as is occasionally done, the proportions can be adjusted readily to either method of mixing.

The following table shows approximately the quantities of cement and sand required for a cubic yard of mortar, of different proportions, mixed in the two ways as above. The quantities in the table are deduced from the results of actual trials; but at best they can be only approximate, since much depends upon the varying conditions of dampness and dryness, looseness and compactness, fineness, degree of burning, etc.

TABLE 11.

INGREDIENTS REQUIRED FOR A GIVEN QUANTITY OF MORTAR OF DIFFERENT PROPORTIONS.

Composition of the Mortar by Volumes.		Cement and Sand required to produce one Cubic Yard of Mortar.					
		Mortar proportioned by volumes of *packed* cement and loose sand.			Mortar proportioned by volumes of *loose* cement and loose sand.		
		Cement, *barrels*.		Sand, *cu. yds.*	Cement, *pounds*.		Sand, *cu. yds.*
Cement.	Sand.	Portland or Ulster Co. Rosendale.	Western Rosendale.		Portland.	Rosendale.	
1	0	7.14	6.43	0.00	2,675	2,140	.00
1	1	4.16	3.74	0.58	1,440	1,150	.67
1	2	2.85	2.57	0.80	900	720	.84
1	3	2.00	1.80	0.90	675	540	.94
1	4	1.70	1.53	0.95	525	420	.98
1	5	1.25	1.13	0.97	425	340	.99
1	6	1.18	1.06	0.98	355	285	1.00

For an example of the method of using this table, see top of page 88.

The left-hand side of the table gives the quantity required when a commercial barrel of cement, *i. e.*, a barrel of packed cement, is mixed with a given number of barrels of sand.

When the cement is shipped in bulk, the right-hand side of the table is to be employed in making estimates. The quantities of cement in this side of the table can be translated into barrels by remembering that the net weight per barrel of cement, although varying somewhat with manufacturers, size of barrels, fineness, etc., is about as follows: Portland, 375 pounds; Eastern Rosendale, 300 pounds; Western Rosendale, 265 pounds.

Cement is also sometimes shipped in bags. Frequently the bags contain an aliquot part of a barrel, in which case either side of the table may be used, according to the method to be employed in mixing the cement and sand. Sometimes the bags contain an even number of hundred-weight, in which case the right-hand side of the table is most convenient.

131. The following data concerning the amount of mortar required per cubic yard for the different classes of masonry, extracted from succeeding pages of this volume, are collected here for greater convenience in making estimates.

TABLE 12.

AMOUNT OF MORTAR REQUIRED FOR A CUBIC YARD OF MASONRY.

DESCRIPTION OF MASONRY.	VOLUME OF MORTAR, *cubic yards.*	
	Min.	Max.
Concrete, broken stone—no screenings or gravel (see §§ 149–51)	0.50	0.55
Rough rubble (§ 214)	0.33	0.40
Rubble with joints rough hammer-dressed	0.25	0.30
Squared stone masonry (see § 210)	0.15	0.20
Ashlar, with 12″ to 20″ courses and ⅜- to ½-inch joints (see § 205)	0.07	0.08
Ashlar, with 20″ to 30″ courses and ¼- to ⅜-inch joints (see § 205)	0.05	0.06
Ashlar, largest blocks and closest joints (see § 205)	0.03	0.04
Brickwork, ⅜- to ½-inch joints (see § 256)	0.35	0.40
" ¼- to ⅜-inch joints " "	0.25	0.30
" ⅛-inch joints " "	0.10	0.15

Example.—How much cement and sand will be required to lay 10 cubic yards of best rubble masonry, using a mortar composed of 1 part packed cement and 3 parts sand? By the preceding table, it is seen that the best rubble will require 0.33 yards of mortar per yard of masonry; hence 10 yards of masonry will require 3.3 cubic yards of mortar. From the table on page 86, it is seen that 1 cubic yard of the above mortar will require 2 barrels of cement and 0.9 cubic yards of sand; hence 3.3 cubic yards of mortar will require 6.6 barrels of cement and 2.97 cubic yards of sand.

132. Strength of Mortar. The strength of mortar is dependent upon the strength of its cementing material, upon the strength of the diluting material, and upon the adhesion of the former to the latter. When sand is used and the grains are entirely enveloped with cementing material, the strength of the mortar will depend upon the strength of the cement and its adhesion; but when the paste is insufficient to fill the voids, the strength of the mortar will depend upon the amount of cementing material and its adhesive power. The strength of the diluting material is an unimportant consideration.

The kind and amount of strength required of mortar depends upon the position in which it is employed. If the blocks are large and well dressed, and if the masonry is subjected to compression only, then the mortar needs only hardness, or the property of resisting compression; but if the blocks are small or irregular, the mortar should have the capacity of adhering to the surfaces of the stone or brick, so as to prevent their displacement; and, if the masonry is liable to be subjected to lateral or oblique forces, the mortar should possess both adhesion and cohesion (tensile strength).

133. Tensile Strength. Fig. 5 shows the effect of time on the tensile strength of cement mortars having different proportions of sand. The diagram was constructed from the "Record of Tests of Cement made for Boston Main Drainage Works." *

"The results were compiled from about 25,000 breakings of twenty different brands, and fairly represent the average strength of ordinary good cements of the two kinds." The diagram is instructive in several ways. It "shows that Portland cement acquires its strength more quickly than Rosendale; that both cements (but especially Rosendale) harden more and more slowly

* E. C. Clarke in Trans. Am. Soc. of C. E., vol. xiv. p. 150.

as the proportion of sand mixed with them is increased; that whereas neat cements and rich mortars attain nearly their ultimate strength in six months or less, weak mortars continue to harden for a year or more. The table shows the advantage of waiting as long as possible before loading masonry structures, and the possibility of saving cost by using less cement when it can have ample

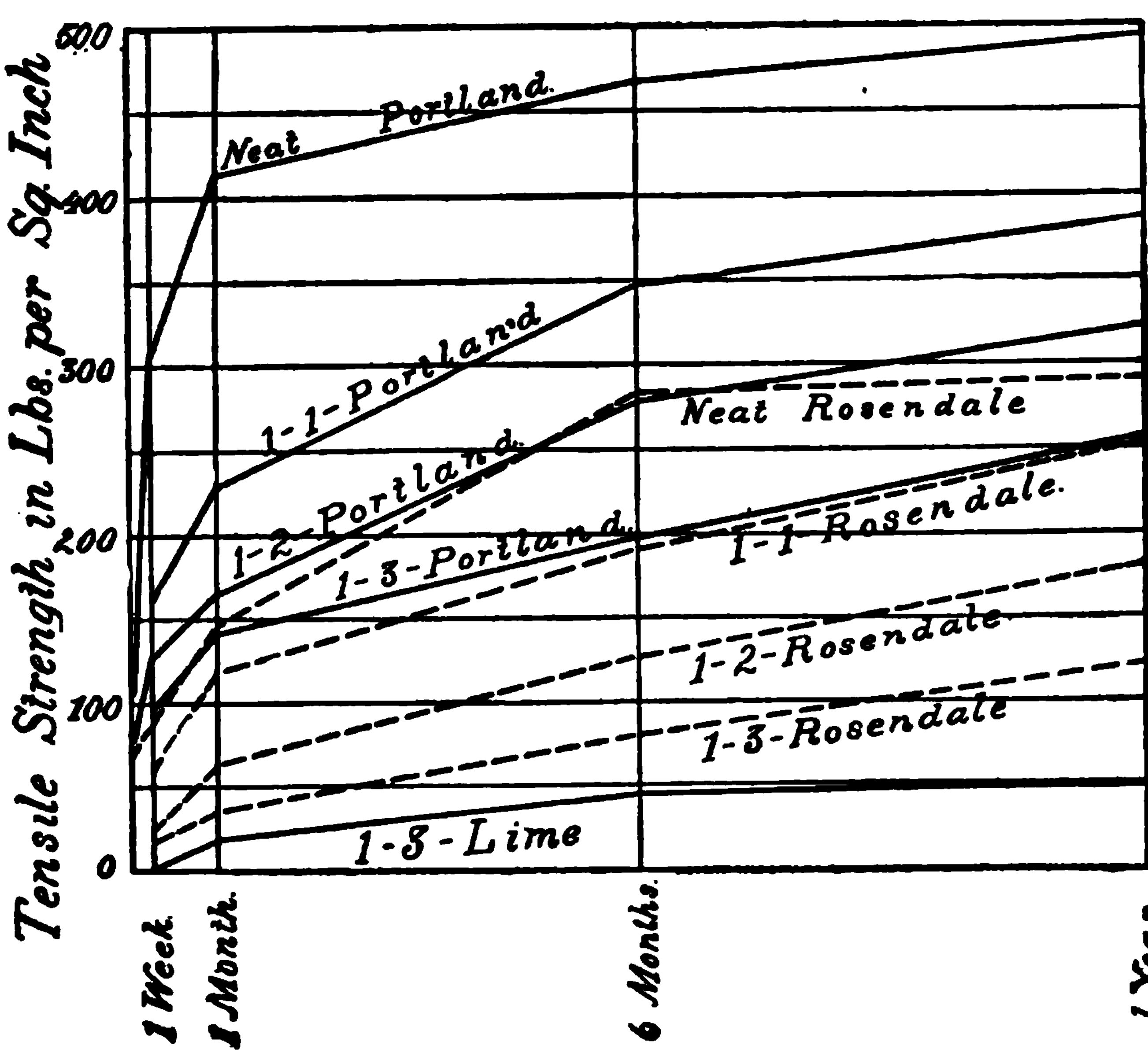

FIG. 5.—DIAGRAM SHOWING THE EFFECT OF TIME ON THE STRENGTH OF CEMENT MORTARS.

time to harden. It also shows that Portland cement is especially useful when heavy strains must be withstood within a week."* The first 24 hours after mixing, Rosendale is the stronger.

The line for the strength of lime mortar probably represents the maximum value that can be obtained by exposing the mortar freely to the air in small briquettes. This line is not well determined.

134. The diagrams of Fig. 6 show the effect of varying the proportions of cement and sand, and also show the relative strength

* E. C. Clarke in Trans. Am. Soc. of C. E., vol. xiv. p. 150.

of Rosendale and Portland mortars at different ages. The diagrams represent the average results of a vast number of experiments. All available data were plotted, and the lines indicating

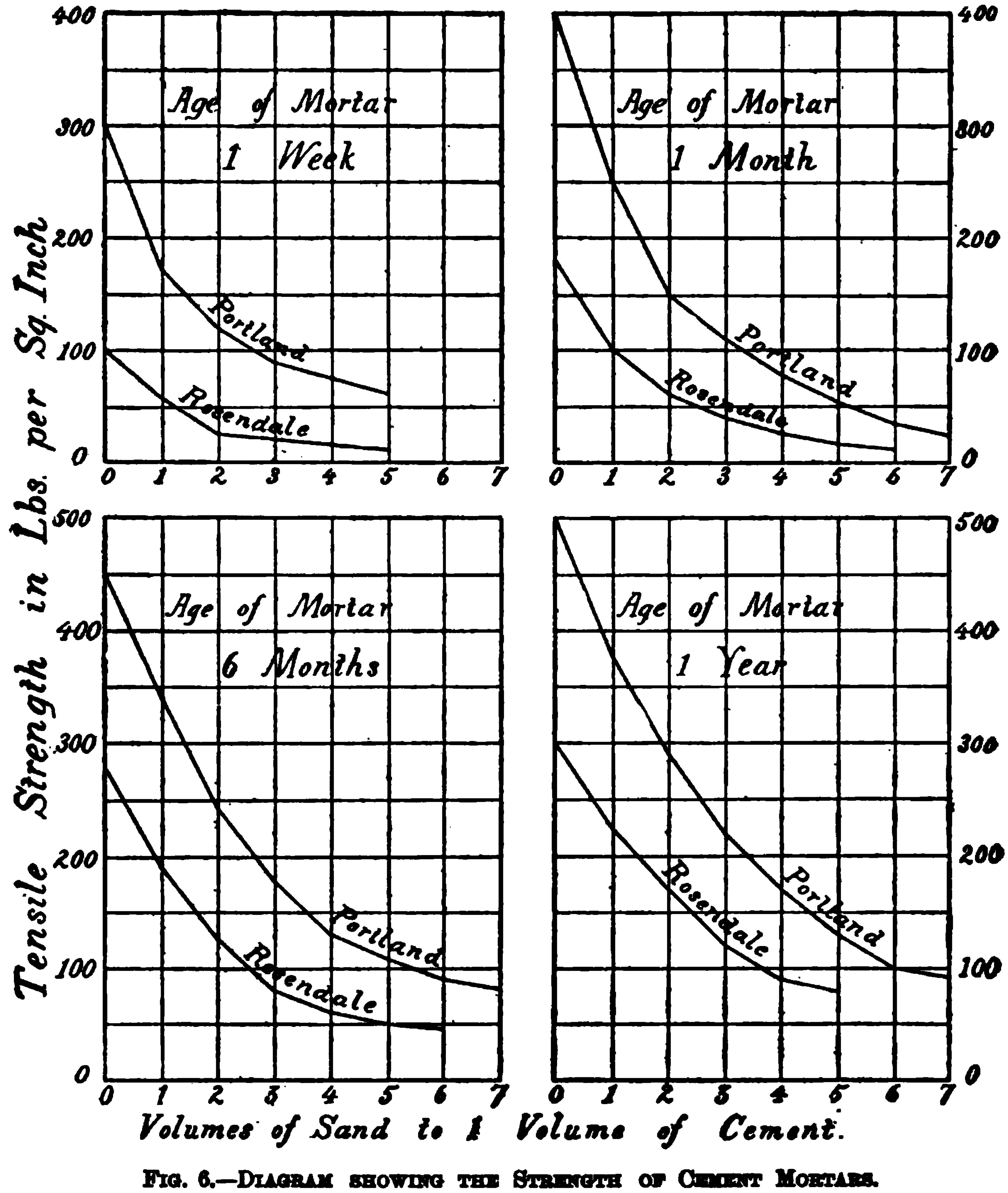

FIG. 6.—DIAGRAM SHOWING THE STRENGTH OF CEMENT MORTARS.

the strength were drawn so as to represent as nearly as possible the mean of all the experiments,—due attention being given to the relative reliability of the different results and to the character of the curve. Since, generally, the individual values plotted were

themselves means, there were no very erratic results, and consequently the lines are quite reliable. There were fewer experiments for the larger proportions of sand to cement; hence the curves are less accurate the larger the proportion of sand. Unusually hard-burned (*i. e.*, heavy) cements, when tested neat, will show a greater strength than that given by the diagram. Very heavy and carefully prepared Portland cements have shown an ultimate strength of 1,000 to 1,200 pounds per square inch. Very fine cements when mixed with sand will show greater strength than that given by the table. The diagrams are intended to give fair working values. These curves are especially useful in discussing the question of economy in the use of cement (see §§ 138–40).

Example.—To illustrate the method of using the diagrams, assume that we desire to know the strength of a 1 to 2 Rosendale cement mortar a year old, and also the proportions of a Portland cement mortar of equal strength. At the bottom of the lower right-hand diagram of Fig. 6 find the proportion of sand in the mortar, which in this case is 2; follow the corresponding line up until it intersects the "Rosendale" line. The elevation of this intersection above the base, as read from the figure at the side of the diagram, is the strength of the specified mixture, which in this case is about 170 pounds per square inch. The second part of the problem, then, is to determine the proportions of a Portland cement mortar which will have a strength of 170 pounds per square inch. Find the 170 point on the scale at the side of the diagram, and imagine a horizontal line passing through this point and intersecting the "Portland" line; from this point of intersection draw a vertical line to the base of the diagram, and this point of intersection gives the required number of volumes of sand to one volume of cement, which in this case is 4. Therefore a 1 to 2 Rosendale mortar a year old has a strength of 170 pounds per square inch, and is then equivalent to a 1 to 4 Portland mortar.

135. Fig. 7 shows the effect of mixing lime and cement. The upper line gives the strength of a mixture of Rosendale cement and lime, and is fairly well determined. The lines showing the strength of a mortar composed of sand, and a paste of lime and cement of the proportions shown at the bottom of the diagram, are drawn on the assumption that adding sand to a paste of cement and lime has the same effect, proportionally, as adding sand to a

paste of neat cement. A few experiments seem to support this conclusion, and none have been made which contradict it.

The addition of lime mortar to cement is more injurious with Portland than with Rosendale. When the lime paste is equal to or exceeds the cement paste, it makes practically no difference in the strength of the mortar which cement is used. For economical considerations, a mixture of Portland cement and lime should not

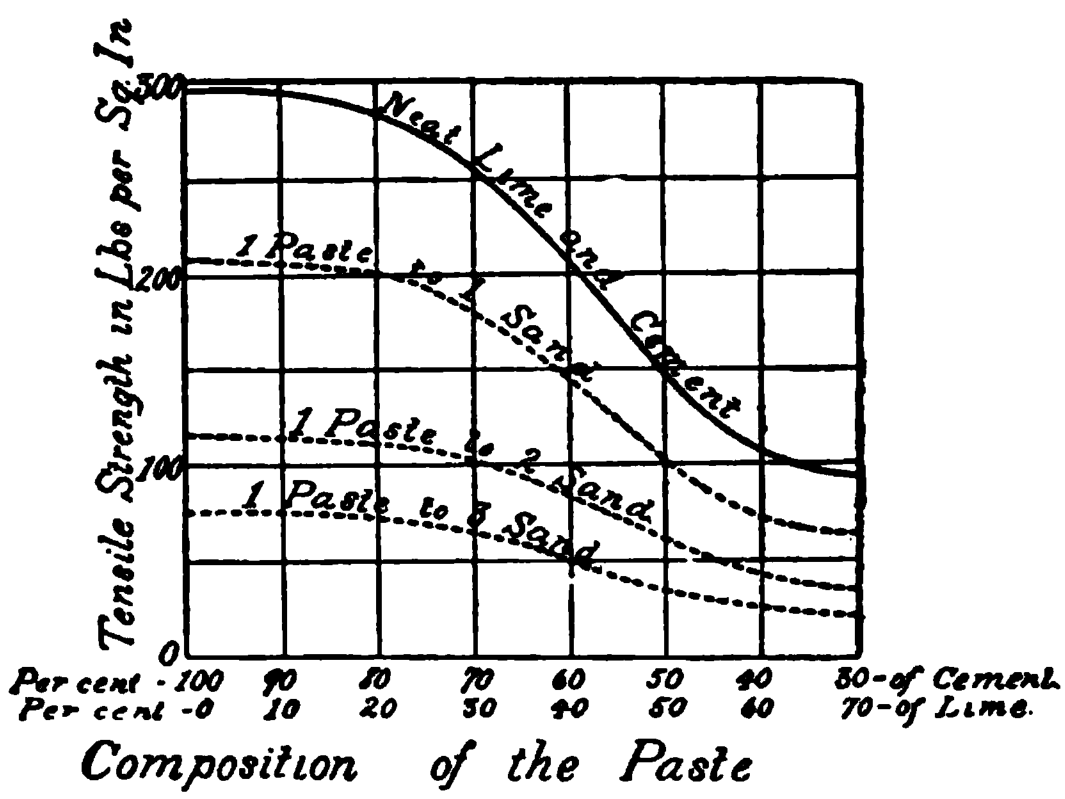

FIG. 7.—DIAGRAM SHOWING THE EFFECT OF ADDING LIME TO CEMENT MORTAR.

be chosen when a mortar intermediate between common lime and cement is desired.

Notice that the addition of a small amount of lime paste has but a slight effect upon the strength of the resulting mortar. An addition of 20 or 25 per cent. of lime will not greatly decrease the strength, but will materially reduce the cost.

136. Compressive Strength. But few experiments have been made upon the compressive strength of mortar. An examination of the results of about sixty experiments made with the Watertown testing-machine seems to show that the compressive strength of mortar, as determined by testing cubes, is from 8 to 10 times the tensile strength (see § 134) of the same mortar at the same age. See also § 97. Data determined by submitting cubes of mortar to a compressive strain are of little or no value as showing the strength of mortar when employed in thin layers, as in the joints of masonry. The strength per unit of bed area increases rapidly as the thickness of the test specimen decreases, but no experiments have ever been made to determine the law of this increase for mortar.

137. Adhesive Strength. Unfortunately very few experiments have been made on the adhesive strength of mortars, *i. e.*, the power with which mortars stick to brick, stone, etc. It is commonly assumed that, after the lapse of a moderate time, the adhesive and cohesive strengths of cement mortars are about equal, and that in old work the former exceeds the latter. Modern experiments, however, fail to establish the truth of this assumption, and indicate rather that the adhesion of mortar to brick or stone is much less, during the first few months, than its tensile strength; and also that the relation between the adhesive strength and cohesive strength (the resistance of the mortar to pulling asunder) is very obscure. The adhesion of mortars to brick or stone varies greatly with the different varieties of these materials, and particularly with their porosity. The adhesion varies also with the quality of the cement, the character, grain, and quantity of the sand, the amount of water used in tempering, the amount of moisture in the stone or brick, and the age of the mortar. Some cements which exhibit high tensile strength give low values for adhesion; and conversely, cements which are apparently poor when tested for cohesion show excellent adhesive qualities.

The table* on the following page gives all the reliable data known. A comparison of the table with the diagram on page 90 shows that the adhesion of a mortar is far less than its tensile strength at the same age, but fails to show any definite relation between the two. In the experiments by Dr. Böhme at the Royal Testing Laboratory at Berlin the mortar was made with standard quartz sand, and the tensile strength of the mortar and its adhesion to common brick were determined separately. By comparing the tensile and adhesive strengths at the same ages, it was found that the former was always about ten times greater than the latter when the mortar consisted of one part of cement to three or four parts of sand, and from six to eight times greater when the cement was used neat or with one or two parts of sand. In the experiments made by Prof. Warren, of Sydney University, New South Wales, the tensile strength of neat Portland cement mortar was three and a half times its adhesion to brick. The result of twelve thousand special tests by Mr. Mann was that pure Portland cement of 425 lbs. tensile and 5,640 lbs.

* Compiled by Emil Kuichling, for the report of the Executive Board of the City of Rochester, N. Y., for 1888.

TABLE 18.—ADHESIVE STRENGTH OF MORTARS.

Reference No.	Age, in days, when tested.	Kind of Cement used.	Materials Cemented together.	Average adhesive strength in pounds per square inch. Neat Cement.	Cement, 1; sand, 1.	Cement, 1; sand, 2.	Cement, 1; sand, 3.	Cement, 1; sand, 4.	Authority.
1	7	Quick-setting cement	H[illegible]	......	......	*25	......	......	Robertson.....1858
2	"	Slow-setting cement	H[illegible]	......	......	*15	......	......	" "
3	"	Portland	S[illegible]	57	......	......	......	......	I. J. Mann....1858
4	"	"	C[illegible]	41	......	......	......	......	" " .... "
5	"	"	P[illegible]	35	......	......	......	......	" " .... "
6	"	"	B[illegible]	19	......	......	......	......	" " .... "
7	"	Hydraulic lime	†[illegible]	34.1	21.0	18.7	15.3	13.2	Dr. Böhme... "
8	"	Portland	‡	165	102	88	20	9	Prof. Warren..'87
9	"	"	§	......	117	53	36	16	" " "
10	16	Quicklime	L[illegible]	......	......	9–15	......	......	Boistard.
11	"	Lime and cement		......	......	5	......	......	"
12	28	Hydraulic lime	†[illegible]	35.1	30.4	25.5	20.9	17.5	Dr. Böhme....1885
13	"	Portland	‡	218	105	45	24	14	Prof. Warren..'87
14	"	"	§	......	146	73	48	45	" " "
15	30	Quick-setting cement	H[illegible]	......	......	*50	......	......	Robertson.....1858
16	"	Slow-setting cement	H[illegible]	......	......	*30	......	......	" "
17	"	Rosendale	C[illegible]	30.6	15.7	12.8	6.3	5.2	Gen. Gillmore.'63
18	"	"	F[illegible]	27.5	20.8	12.0	9.2	7.9	" " "
19	"	Portland	S[illegible]	78	......	......	......	......	I. J. Mann....1883
20	"	"	Cut granite	67	......	......	......	......	" " .... "
21	"	"	Polished marble	71	......	......	......	......	" " .... "
22	"	"	Bridgewater brick	105	......	......	......	......	" " .... "
23	"	"	Sandstone	40	......	......	......	......	" " .... "
24	"	Blue lias lime	Staffordshire brick	......	......	*40	......	......	*Building News*,'80
25	"	" " "	Gray stock brick	......	......	*36	......	......	" " "
26	"	" " "	Common soft "	......	......	*18	......	......	" " "
27	"	Lime and pozzuolana	Hard brick	......	......	*5	......	......	J. White.....1882
28	42	Portland	¶Brick	62.5	......	46.9	......	......	Bauschinger..1873
29	49	"	¶ "	......	......	......	34.2	......	" "
30	56	"	¶ "	......	54.0	54.9	......	......	" "
31	90	Hydraulic lime	† "	39.8	41.9	36.9	28.1	22.6	Dr. Böhme....1888
32	95	Portland	¶ "	......	......	......	14.2	......	Bauschinger..1873
33	110	Hydraulic lime	¶ "	......	......	......	12.8	......	" "
34	180	Quicklime	Brick	......	......	*28	......	......	Rondelet.....1881
35	"	"	Limestone	......	......	*15	......	......	" "
36	"	"	Hard brick	......	......	*40	......	......	Robertson....1858
37	"	"	Soft brick	......	......	*18	......	......	" "
38	"	Portland	Sawed slate	160	......	......	......	......	I. J. Mann....1883
39	"	"	Portland stone	155	......	......	......	......	" " .... "
40	"	"	Polished marble	175	......	......	......	......	" " .... "
41	270	Lime and pozzuolana	Hard brick	......	......	*8	......	......	J. White.....1882
42	320	Rosendale	Croton brick	66	40	24	......	......	Gen.Gillmore 1863
43	1 yr	Quicklime	Not stated	......	......	......	*21	....	Vicat..........1818
44	"	Good quicklime	" "	......	......	......	*51	....	" .... "
45	"	Ordinary hydraulic lime	" "	......	......	*85	......	......	" .... "
46	"	Good hydraulic lime	" "	......	......	*140	......	......	" .... "
47	"	" " "	Materials in air	......	70	......	......	......	Mallet.........1839
48	"	" " "	" in water	......	93	......	......	......	" .... "
49	"	Portland	Gault-clay brick pressed	45	44	......	......	......	J. Grant......1871
50	"	"	Stock brick in air	78	68	......	......	......	" ...... "
51	"	"	Stock brick in water	96	[illegible]	......	......	......	" ...... "
52	"	"	Staf. blue brick in air	46	47	......	......	......	" ...... "
53	"	"	Staf. blue brick in water	40	29	......	......	......	" ...... "
54	"	"	Fareham red brick in air	126	88	......	......	......	" ...... "
55	"	"	Fareham red brick in water	138	62	......	......	......	" ...... "

* Proportions of sand not given, but presumably about those indicated in headings of table.
† Standard sand used in mixture. ‡ Clean river-sand used in mixture.
§ Crushed sandstone used in mixture.
‖ Coarse particles in cement sifted out before testing. ¶ Fine river-sand used in mixture.

compressive strength per square inch has but 60 to 80 lbs. adhesion to limestone, and that the ratio of tensile to adhesive strength varies from 5.1 to 9.1.

138. Cost of Mortar. The analysis of the cost of a cubic yard of mortar composed of 1 volume of cement (packed) to 2 volumes of sand (loose) is about as follows:

Cement	2.85 bbls. (see page 86) @ \$1.20 =	\$3 43
Sand	0.8 cu. yd. (see page 86) @ 0.40 =	32
Labor, handling materials and mixing	⅓ day @ 1.50 =	50
		\$4 25

Of course the cost will vary with the composition of the mortar, but with the table on page 86 and the current prices of materials, it is easy to compute the cost. The expense for labor is quite variable, depending upon the distance the ingredients must be moved, etc. If the mixing is done by machinery, the cost of mixing may be as low as 25 cents per cubic yard.

139. It is frequently a question as to whether Portland or Rosendale cement is the more economical. This is a question of relative strengths and relative prices. The diagrams of Fig. 6 show the relative strength, at different ages, of Rosendale and Portland mortars of varying proportions of cement and sand.

To illustrate the method of using these diagrams, let it be assumed that a mortar is desired which shall have a strength of 90 to 100 lbs. at the end of a week. What composition shall the mortar have, and which cement is the cheaper? By the upper left-hand diagram it is seen that a clear Rosendale and also a 1 to 3 Portland mortar possesses the requisite strength. The cost of a cubic yard of each—if it be assumed that both the cement and sand are measured loose, *i. e.*, if we use the right-hand side of Table 11—will be about as follows:

Rosendale, neat.	
Cement (2,140 lbs.), 7.14 bbls. @ \$1.00	\$7 14
Labor	50
Total cost	\$7 64

Portland, 1 to 3.	
Cement (675 lbs.), 1.8 bbls. @ \$3.00	\$5 40
Sand, 0.94 yd. @ \$0.40	37
Labor	50
Total cost	\$6 27

Again, at the end of one year a 1 to 2 Rosendale mortar is equal in strength to a 1 to 4 Portland mortar. The cost of these mortars is as follows:

Rosendale, 1 to 2.		*Portland*, 1 to 4.	
Cement (720 lbs.), 2.40 bbls. @ $1.00	$2 40	Cement (525 lbs.), 1.4 bbls. @ $3.00	$4 20
Sand, 0.84 cu. yd. @ $0.40....	33	Sand, 0.98 cu. yd. @ $0.40....	40
Labor........................	50	Labor........................	50
Total cost........	$3 23	Total cost........	$5 10

If, in the mixing, the cement be measured in its compacted state, *i. e.*, if the left-hand side of Table 11 be used, the cost for the first mortar in the above examples will be $7.64 and $6.86, respectively; under the same conditions the second mortar will cost $3.67 and $5.98, respectively.

140. It will be noticed in the above examples that in the first case the Portland is the cheaper, and in the second the Rosendale. However, outside of any question of strength or first cost, the slow-setting property of Portland cement gives it a decided advantage over Rosendale, as has already been discussed (§ 126).

When a very cheap mortar is desired, it is customary to use cement with a large dose of sand; but this is objectionable, since the resulting mortar is very porous and consequently disintegrates rapidly. It is better to add common lime to the cement rather than to increase the quantity of sand. This is, then, a question of relative strengths and relative cost, and sometimes also of relative activity. The diagram on page 92 shows the strength of various mortars with varying proportions of cement and lime, from which the strength of any proposed combination can be determined. The amount of the several ingredients required can be determined by the table on page 86, having which the cost can easily be computed. Gen. Gillmore frequently urges the economy of mixing lime paste with cement mortar.

141. Mortar Impervious to Water. Nearly every failure of masonry is due to the disintegration of the mortar in the outside of the joints. Ordinary mortar—either lime or cement—absorbs water freely, common lime mortar absorbing from 50 to 60 per cent. of its own weight, and the best Portland cement mortar from 10 to 20 per cent.; and consequently they disintegrate under the action

of the frost. Mortar may be made practically non-absorbent by the addition of alum and potash soap. One per cent., by weight, of powdered alum is added to the dry cement and sand, and thoroughly mixed; and about one per cent. of any potash soap (ordinary soft-soap made from wood ashes is very good) is dissolved in the water used in making the mortar. The alum and soap combine, and form compounds of alumina and the fatty acids, which are insoluble in water. These compounds are not acted upon by the carbonic acid of the air, and add considerably to the early strength of the mortar, and somewhat to its ultimate strength.

With lime mortar, the alum and soap has a slight disadvantage in that the compounds which render the mortar impervious to water also prevent the air from coming in contact with the lime, and consequently prevent the setting of the mortar. On the other hand, the alum and soap compounds add considerably to both the early and the ultimate strength of the mortar.

This method of rendering mortar impervious is an application of the principle of Sylvester's method of repelling moisture from external walls by applying alum and soap washes alternately on the outside of the wall (see § 263). The same principle is applied in McMurtrie's artificial stone (see § 162). The alum and soap are easily used, and do not add greatly to the cost of the mortar. The mixture could be advantageously used in plastering, and in both cement and lime mortars of outside walls or masonry in damp places. It has been very successfully used in the plastering of cellar and basement walls. It should be employed in all mortar used for pointing (§ 204).

The addition of a small amount of finely powdered clay decreases the permeability of mortar (see § 117); but since clay absorbs and parts with water with the changing seasons, the use of clay is not efficient in preventing disintegration by freezing and thawing.

142. Sugar in Mortar. Although saccharine matter has been employed in India as an ingredient of mortar from time immemorial, and reference has been made to it by standard authorities, its effect is not generally known; and the subject has attracted considerable attention in England and America during the past year.

Sugar unites with lime and forms sucrate of lime,—a solid which possesses considerable strength, dissolves freely in water, and is acted upon by carbonic acid. All hydraulic cements contain at least 50

per cent. of lime compounds; hence, if a saccharine substance be added to mortar, the sugar will unite with the lime and form sucrate of lime. The presence of this compound may be an advantage or a disadvantage, according to attendant conditions. For example, if the mortar is composed of common lime and sand, the sucrate of lime, being stronger than the carbonate, will add to the strength of the mortar; and as the lime will unite with the sugar more rapidly than with the carbonic acid of the air, the sugar will also cause the mortar to set more quickly. In India, the practice is to add 1 pound of the coarsest sugar (or its equivalent in syrup) to each gallon of water with which the mortar is mixed. This amount of sugar is said to add one half to the breaking strength of the mortar and to double its cohesive strength. It is better to dissolve the sugar in the water than to mix it dry with the lime, since some limes in slaking "burn" the sugar, thereby destroying its strengthening effect and also blackening the mortar.

The addition of sugar increases the early strength of lime mortar, since the sucrate of lime develops its strength more rapidly than the carbonate. If lime mortar were used in the interior of thick walls, the addition of a saccharine substance would be beneficial, since lime mortar thus placed would never become fully saturated with carbonic acid.* The addition of sugar to cement mortar will accelerate or retard the setting of the cement, depending upon (1) the amount of sugar present (as far as the cement is concerned the sugar is an adulteration), (2) the relative indurating activity of the sucrate and the silicate, and (3) the amount of water used (the cement is hydraulic while the sucrate is non-hydraulic, and hence the former will set in the presence of water while the latter will not). This principle may explain the conflicting results obtained from different experiments. For example, one experimenter † found that sugar greatly accelerates the setting of Portland cement, causing it to set in a few minutes; on the other hand, most experimenters ‡ find that sugar in any proportions retards the setting of Portland cement.

* Lime mortar has been taken from the walls of ancient buildings which were only 50 to 80 per cent. saturated with carbonic acid after an exposure of nearly 2,000 years.

† *Engineering News*, vol. xvii. p. 6.

‡ For example, *Mechanics*, vol. ix. pp. 315–17;—a paper read at the Washington meeting of the Am. Soc. of M. E., to be published in vol. ix. of the Transactions of that Society.

All experimenters agree that sugar retards the setting of Rosendale cement.

Sugar added to mortar may decrease or increase the ultimate strength of the mortar, depending upon (1) the amount of sugar present and (2) the relative ultimate strength of the compounds formed. For example, with lime mortar the maximum effect—an addition of 50 per cent. to the ultimate tensile strength—is obtained when the weight of the sugar is equal to about 10 per cent. of the weight of the lime. With neat Rosendale cement mortar, the maximum effect at the end of three months—an addition of about 20 per cent. to the tensile strength—is obtained with $\frac{1}{8}$ to $\frac{1}{4}$ per cent. of sugar. With neat Portland cement mortar the evidence is conflicting. One experimenter * obtained a maximum effect—an addition of 25 per cent. to the strength—with 1 per cent. of sugar; while another † concluded that "sugar was of no great advantage in combination with the best qualities of Portland cement." The last conclusion was corroborated by experiments made by the author.

143. The sucrate of lime, being soluble in water, will in time be washed out by the rain; therefore the addition of a saccharine substance to mortar is most beneficial in a dry climate, as in India, for example. A saccharine substance should not be added when the cement is to be used under water. The compounds of lime with sugar are attacked by the carbonic acid of the air, and hence the strengthening effect of the sugar is not permanent when the mortar is exposed to the weather. Owing to these two facts, the use of sugar with cement is not of much practical value. Although sugar adds materially to the strength of lime mortar, the compound is inferior in strength, cost, and durability to cement mortar.

It is highly probable that the effects obtained by mixing sugar with mortar can also be obtained by the use of gum-arabic, dextrine, glucose, or starch. The use of such materials in mortar involves some interesting questions; and a study of this subject by an engineer-chemist might lead to valuable results.

Plaster of Paris (sulphate of lime) added to either lime or cement mortar has the same general effect as adding sugar; that is to say, added in quantities not exceeding 5 per cent. it accelerates the

* *Mechanics*, vol. ix. pp. 315–17; a paper read at the Washington meeting of the Am. Soc. of M. E., to be published in vol. ix. of the Transactions of that Society.

† *Engineering News*, vol. xvi. p. 333.

setting, and also increases the early and the ultimate strength of mortar. The addition of plaster of Paris to common cement is the special feature of selenitic cement, which is much used in England.

144. Freezing of Mortar. It does not appear that common *lime mortar* is seriously injured by freezing, provided it remains frozen until it has fully set. The freezing retards, but does not entirely suspend, the setting. Alternate freezing and thawing materially damages the strength and adhesion of lime mortar. Although the strength of the mortar is not decreased by freezing, it is not always permissible to lay masonry during freezing weather; for example, if, in a thin wall, the mortar freezes before setting and afterward thaws on one side only, the wall may settle injuriously.

"Mortar composed of 1 part *Portland cement* and 3 parts of sand is entirely uninjured by freezing and thawing. Mortar made of *cements* of the *Rosendale type*, in any proportion, is entirely ruined by freezing and thawing."* It is probable that this difference is due to the fact that the expansive force of water in freezing is less than the strength of the richer Portland cement mortars and greater than any Rosendale cement mortar. In the latter case, freezing is equivalent to agitating the mortar after it has partially set. Mortar made of over-clayed cement (which condition is indicated by its quicker setting), of either the Portland or Rosendale type, will not withstand the action of frost as well as one containing less clay; for since the clay absorbs an excess of water, it gives an increased effect to the action of frost.

It is customary, in making cement mortar which is liable to be frozen before setting, to add salt to the water with which it is mixed. The ordinary rule is: "Dissolve 1 pound of salt in 18 gallons of water when the temperature is at 32° Fahr., and add three ounces of salt for every 3 degrees of lower temperature." Since the salt contains carbonic acid, and since lime mortar hardens by the absorption of this acid, it would seem that salt should strengthen lime mortar; but of this there is no experimental proof. The use of salt, and more especially of sea water, in mortar is objectionable, since the accompanying salts usually produce efflorescence (see § 265). It is also probable that fresh water charged with carbonates makes slightly stronger mortar than pure water. It is not certain that the addition of salt to Portland cement mortar does either any good or any

* Trans. Am. Soc. of C. E., vol. xvi. pp. 79–84.

harm; it appears to retard slightly the setting.* Rosendale cement mortar, when mixed with a saturated solution of salt and water, does not freeze at the ordinary winter temperature of New York; † and hence it is not injured by frost.

When masonry is to be laid in lime mortar during freezing weather, frequently the mortar is mixed with a minimum of water and then thinned to the proper consistency by adding hot water just before using. This is undesirable practice (see § 120). When the very best results are sought, the brick or stone should be warmed —enough to thaw off any ice upon the surface is sufficient— before being laid. They may be warmed either by laying them on a furnace, or by suspending them over a slow fire, or by wetting with hot water.

145. Change of Volume in Setting. The Committee of the American Society of Civil Engineers draw the following conclusions: ‡ 1. Cement mortars hardening in air diminish in linear dimensions, at least to the end of twelve weeks, and in most cases progressively. 2. Cement mortars hardening in water increase in like manner, but to a less degree. 3. The contractions and expansions are greatest in neat cement mortars. 4. The quick-setting cements show greater expansions and contractions than the slow-setting cements. 5. The changes are less in mortars containing sand. 6. The changes are less in water than in air. 7. The contraction at the end of twelve weeks is as follows: for neat cement mortar, 0.14 to 0.32 per cent.; for a mortar composed of 1 part cement and 1 part sand, 0.08 to 0.17 per cent. 8. The expansion at the end of twelve weeks is as follows: for neat cement, 0.04 to 0.25 per cent.; for 1 part cement and 1 part sand, 0.0 to 0.08 per cent. 9. The contraction or expansion is essentially the same in all directions.

146. Elasticity, Compression, and Set of Mortar. For data on elasticity see page 13. The evidence is so conflicting that it is impossible to determine the co-efficient of compression and of set of mortar, even approximately. For much valuable data on this and related subjects, see the "Report of Progress of the Committee

* Trans. Am. Soc. of C. E., vol. xvi. p. 81.

† Ibid. p. 83.

‡ See the "Report of Progress of the Committee on the Compressive Strength of Cements and the Compression of Mortars and Settlement of Masonry," in the Transactions of that Society, vol. xvii. pp. 213-37; also a similar report in vol. xvi. pp. 717-32.

on the Compressive Strength of Cements and the Compression of Mortars and Settlement of Masonry," in the Transactions of the American Society of Civil Engineers, vol. xvi. pp. 717–32, vol. xvii. pp. 213–17, and also vol. xviii. pp. 264–80. The several annual reports of tests made with the United States Government testing-machine at Watertown contain valuable data—particularly the report for 1884, pp. 69–247—bearing indirectly upon this and related subjects; but since some of the details of the experiments are wanting, and since the fundamental principles are not well enough understood to carry out intelligently a series of experiments, it is impossible to draw any valuable conclusions from the data.

ART. 2. CONCRETE.

147. Concrete is a species of artificial stone. It is sometimes called *béton*, the French equivalent of concrete.

"Concrete is admirably adapted to a variety of most important uses. For foundations in damp and yielding soils and for subterranean and submarine masonry, under almost every combination of circumstances likely to be met with in practice, it is superior to brick masonry in strength, hardness, and durability; is more economical; and in some cases is a safe substitute for the best natural stone, while it is almost always preferable to the poorer varieties. For submarine masonry, concrete possesses the advantage that it can be laid, under certain precautions, without exhausting the water and without the use of a diving-bell or submarine armor.* On account of its continuity and its impermeability to water it is an excellent material to form a substratum in soils infested with springs; for sewers and conduits; for basement and sustaining walls; for columns, piers, and abutments; for the hearting and backing of walls faced with bricks, rubble, or ashlar work; for pavements in areas, basements, sidewalks, and cellars; for the walls and floors of cisterns, vaults, etc. Groined and vaulted arches, and even entire bridges, dwelling-houses, and factories, in single monolithic masses, with suitable ornamentation, have been constructed of this material alone." †

* For a series of papers on Concrete as Applied to Harbor Improvements, see Proc. Inst. of C. E., vol. lxxxvii. pp. 65–240.

† Gillmore's Limes, Hydraulic Cements, and Mortars, p. 225.

The great value of concrete in all kinds of foundations is slowly coming to be appreciated. It is superior to any other form of base. It enables the engineer to build his superstructure on a monolith as long, as wide, and as deep as he may think best, which cannot fail in parts, but, if rightly proportioned, must go all together—if it fails at all.

148. Ingredients. Concrete is composed of (1) the matrix, which may be either lime or cement mortar—usually the latter,—and (2) the aggregate, which may be any hard material, as gravel, pebbles, broken shells, stone, broken brick, slag, etc.

The mortar may be made as already described in Art. 1 above. Whatever the aggregate, it should be free from dust, earth, or any weak material. The aggregate should be of different sizes, so that the smaller shall fit into the voids between the larger. This requires less cement, and with good aggregate gives a stronger concrete. Porous substances make the best aggregates, as the mortar does not stick well to hard, smooth surfaces. Broken stone is the most common aggregate. It is usual to require that the stone shall be broken in pieces so as to pass, any way, through a 2-inch ring. The proportion of matrix should slightly exceed the voids in the aggregate.

The proportion of voids may be determined by experiment in either of the following ways: 1. Determine the specific gravity of the solid aggregate, and from that the weight of a unit of volume of the solid. Weigh a unit of volume of the loose material. The difference between these weights divided by the first gives the proportion of voids. 2. Wet the loose aggregate thoroughly; fill a vessel of known capacity with it; and then pour in all the water the vessel will contain. Measure the volume of water required, and divide this by the volume of the vessel; the quotient represents the proportion of voids.

The voids of broken stone, in which the size and shape of the pieces are nearly uniform, are about 0.5 of the mass. If the pieces are not uniform, the voids are about 0.4 of the mass. The voids in gravel vary, but average about 0.5 of the mass.

149. Proportions. * The usual proportion for concrete is 1 volume of cement mortar to 4 or 5 volumes of broken stone. 1 or 2 volumes of gravel is sometimes added, which decreases the amount of mortar required, and also increases the strength of the concrete.

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* See also § 158.

The following proportions will make a first-class Rosendale concrete, and one suitable for use wherever a strong concrete is required. Of course, if Portland cement were substituted for the Rosendale the concrete would be still better; but this would rarely be done, owing to the greater expense. The stone should be broken so as to pass, any way, through a $2\frac{1}{2}$-inch ring.

$$\left.\begin{array}{ll}2 & \text{bbls. (300 lbs. each) of cement} \\ 4 & \text{'' (0.5 cu. yd.) of sand}\end{array}\right\} = 0.55 \text{ cu. yd. of mortar.}$$

0.9 cu. yd. of broken stone.

1 cu. yd. of concrete.

The following concrete is considerably cheaper and only a little less strong than the preceding:

$$\left.\begin{array}{ll}1 & \text{bbl. (300 lbs.) of cement} \\ 2 & \text{bbls. (0.25 cu. yd.) of sand}\end{array}\right\} = 0.28 \text{ cu. yd. of mortar.}$$

0.5 cu. yd. of gravel.
0.9 " of broken stone.

1 cu. yd. of concrete.

150. According to Gen. Gillmore, the following is the formula for the concrete used in the foundations built by the U. S. army engineers:

$$\left.\begin{array}{l}\text{1 bbl. (300 lbs.) of cement} \\ \text{3 bbls. of sand}\end{array}\right\} = 3.27 \text{ bbls. of mortar.}$$

5 " of broken stone, gravel, broken brick, etc.

0.8 cu. yd. of concrete rammed in place.

The composition of a yard of the above concrete is as follows:—

$$\left.\begin{array}{l}\text{1.25 bbls. of cement} \\ \text{.50 cu. yd. of sand}\end{array}\right\} = 0.56 \text{ cu. yd. of mortar.}$$

.90 " of broken stone.

1.00 cu. yd. of concrete rammed in place.

For a cheaper concrete, Gen. Gillmore recommends the addition of another barrel or barrel and a half of broken stone to the above mixture; and also, if a still cheaper concrete is desired, the use of the following formula:

1 bbl. (300 lbs.) of Rosendale cement = 3.7 cu. ft. paste.
0.33 " of common lime = 2.5 " "
3.5 to 4 bbls. of loose sand.

5 cu. ft. of mortar.

The concrete consists of 1 volume of this mortar to 2½ volumes of ballast. It is desirable in all cases that the mortar for concrete should be hydraulic, in order to secure simultaneous induration throughout the entire mass. If this concrete is laid in large masses or under water, the lime paste is only equivalent to its bulk of inert material.

151. In building the Mississippi jetties, blocks of concrete were used which weighed from 25 to 72 tons each. The materials in a cubic yard of the concrete after it had set were as follows:

Portland cement	0.16	cubic yard.
Sand	0.45	" "
Clean gravel	0.24	" "
Broken stone	0.81	" "
Total materials	1.66	cubic yards.

The concrete after setting was only 60 per cent. of the volume of the dry materials. The contraction during induration was 4 per cent.*

152. MIXING. The concrete may be mixed by hand or by machinery.

In the first method, the cement and sand are mixed as described in the second paragraph of § 123. A basin is then formed by drawing the sand and cement to the outer edges of the box, and the water is poured into it. The sand and cement are then thrown back upon the water, the whole mass thoroughly mixed with the hoe or shovel, and then leveled off. The broken stone should be sprinkled with sufficient water to remove all dust and thoroughly wet the entire surface; the amount of water required will vary considerably with the absorptive power of the stone and the temperature of the air. The wet stone is then to be spread evenly over the top of the mortar, and the whole mass thoroughly mixed by turning up with a shovel.

When large quantities of concrete are required, it is mixed by machinery. One of the simplest machines for this purpose is a spiral conveyer running in a trough into which the ingredients are shoveled. A common kind consists of a box or cylinder to receive the ingredients, revolving slowly about a diagonal or eccentric axis. In another form, the sand, cement, and water are mixed by an end-

* Corthell's Jetties of the Mississippi River.

less screw working in an inclined cylinder; and the mortar and stone then are mixed by another similar contrivance. In some of the machines the proportions of the ingredients are measured by the machine itself.

153. Laying. After mixing, the concrete is conveyed in wheelbarrows or in buckets swung from a crane, and compacted in position by ramming in layers 6 to 8 inches thick. Concrete should not be mixed with too much water, but when ready for use should be quite coherent and capable of standing at a steep slope without the water running from it; otherwise it will be impossible to compact it by ramming. It should not be plastic and jelly-like under the rammer, and the ramming should continue only until the water begins to ooze out on the upper surface. Too severe or long-continued pounding injures the strength of the concrete by forcing the broken stone to the bottom of the layer, and by disturbing the incipient set of the cement.

The rammer generally consists of a log of wood with a handle attached at one end, and is operated by two men.

When one layer is laid on another already partially set, the entire surface of the latter should be thoroughly wet. When concrete is in place, there should be no walking on it for at least 12 hours.

154. Depositing Concrete under Water. In laying concrete under water, an essential requisite is that the materials shall not fall from any height, but be deposited in the allotted place in a compact mass; otherwise the cement will be separated from the other ingredients and the strength of the work be seriously impaired. If the concrete is allowed to fall through the water, its ingredients will be deposited in a series, the heaviest—the stone—at the bottom and the lightest—the cement—at the top, a fall of even a few feet causing an appreciable separation. Of course concrete should not be used in running water, as the cement would be washed out.

A common method of depositing concrete under water is to place it in a V-shaped box of wood or plate-iron, which is lowered to the bottom by a crane. The box is so constructed that, on reaching the bottom, a pin may be drawn out by a string reaching to the surface, thus permitting one of the sloping sides to swing open and allowing the concrete to fall out. The box is then raised to be refilled. It usually has a lid. Concrete under water should not be

rammed; but, if necessary, may be leveled by a rake or other suitable tool immediately after being deposited.

A long box or tube, called a *trémie*, is also sometimes used. It consists of a tube open at top and bottom, built in detachable sections so that the length may be adjusted to the depth of water. The tube is suspended from a crane, or movable frame running on a track, by which it is moved about as the work progresses. The upper end is hopper-shaped, and is kept above the water; the lower end rests against the bottom. The trémie is filled in the beginning by placing the lower end in a box with a movable bottom, filling the tube, lowering all to the bottom, and then detaching the bottom of the box. The tube is kept full of concrete, as the mass issues from the bottom more being thrown in at the top.

Concrete has also been successfully deposited under water by enclosing it in paper bags, and lowering or sliding them down a chute into place. The bags get wet and the pressure of the concrete soon bursts them, thus allowing the concrete to unite into a solid mass. Concrete is also sometimes deposited under water by enclosing it in open-cloth bags, the cement oozing through the meshes sufficiently to unite the whole into a single mass.

When concrete is deposited in water, a pulpy gelatinous fluid is washed from the cement and rises to the surface. This causes the water to assume a milky hue; hence the term *laitance*, which French engineers apply to this substance. It is more abundant in salt water than in fresh water. It sets very slowly, and sometimes scarcely at all, and its interposition between the layers of concrete forms strata of separation. The proportion of laitance is greatly diminished by using large immersing boxes, or a trémie, or paper or cloth bags.

155. STRENGTH OF CONCRETE. The strength of concrete depends upon the kind and amount of cement, and upon the kind, size, and strength of the ballast. It is clear that the mortar will adhere to broken stone better than to pebbles, and that therefore concrete containing the former is stronger than that containing the latter. It is also clear that if the sizes of the individual pieces of the ballast are so adjusted that the smaller fit into the interstices of the larger, successively, then the cementing material will act to the best advantage and consequently the concrete will be stronger. Ramming the concrete after it is in place brings the pieces of ballast into closer contact, and consequently makes it stronger. If the bond

is equally good, the larger the pieces of ballast the better. The strength of concrete also depends somewhat upon the strength of the ballast; but since the strength of the concrete depends almost entirely upon the adhesion of the mortar to the ballast, the strength of the latter is not an important element. But few experiments have been made to determine the strength of concrete; and the meager results are very discordant, owing, doubtless, to the varying conditions of the experiments.

156. Transverse Strength. Fig. 8 shows the results of all the experiments that could be found which were definite or without internal evidences of unreliability. The data represented by the

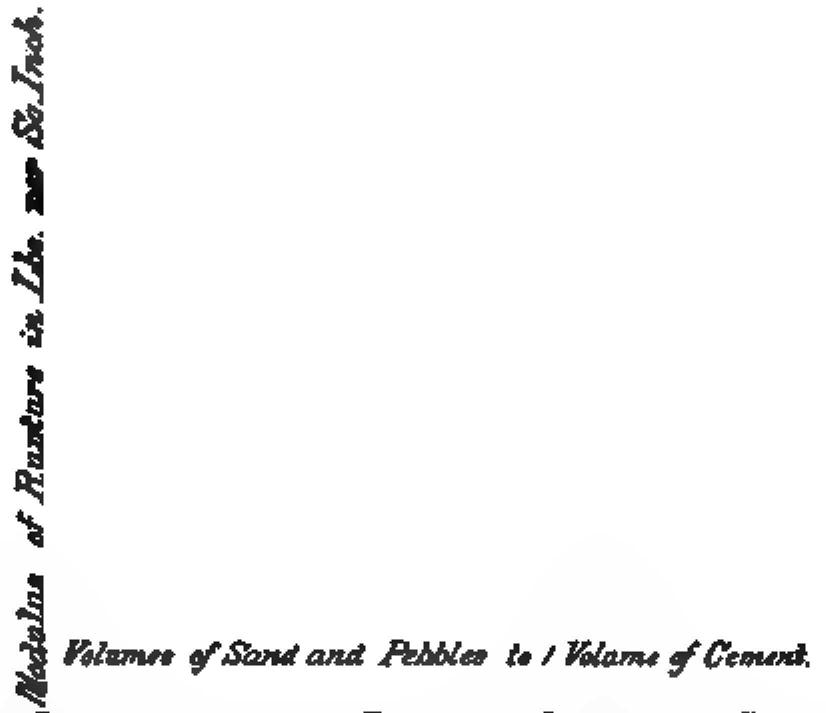

FIG. 8.—DIAGRAM SHOWING THE TRANSVERSE STRENGTH OF CONCRETE.

three lines marked Portland were computed from the results of fifty-five experiments made at Boulogne-sur-Mer by Voisin, published in *Annales des Ponts et Chaussées* for 1858. The mortar was composed of natural Portland cement from Boulogne-sur-Mer, and sand, the proportions varying from 1 to 1 to 1 to 6. The ballast consisted of pebbles. Other experiments seem to show that Boulogne cement is a little stronger than the average of Portland cement; therefore the results were decreased 10 per cent. before being plotted. The values, up to a proportion of 1 cement to 2 diluting material, were determined by experiments upon ordinary mortars. The diagram agrees fairly well with the results of twelve experiments by Gen.

Gillmore, as given in his treatise on "Coignet Béton and other Artificial Stones."

The line marked Rosendale was interpolated, and represents the probable strength of concrete composed of Rosendale cement, sand, and pebbles in the proportion shown. The three points marked × indicate the strength of concrete 6 months old, composed of Portland cement, sand, and pebbles 1 inch or less in diameter; and the point marked * indicates the strength of Rosendale concrete under the same conditions.† Each of these points represents a single experiment.

157. Compressive Strength. Trautwine says‡ that cubes of Portland cement, sand, and broken stone, "well made and rammed, should, either in air or in water, require to crush them at different ages not less than about as follows:

Age in months............	1	3	6	9	12
Tons per sq. ft	15	40	65	85	100

Under favorable conditions of materials, workmanship, and weather, the strengths may be from 50 to 100 per cent. greater."

The compressive strength of 6-inch cubes of concrete, exposed to the air for six months, as determined in connection with the construction of the St. Louis Bridge, was as follows: § With the proportions of 1 part cement (Akron and Louisville), 1 part sand, and 4 parts broken limestone, the mean compressive resistance from 9 trials was 1,200 lbs. per sq. in. (85 tons per sq. ft.); and with the proportions of 1, 2, 4, respectively, the average resistance from 12 trials was 940 lbs. per sq. in. (70 tons per sq. ft.).

Tests with the United States testing-machine ‖ at Watertown, Mass., between steel, gave an average of 1,544 lbs. per sq. in. (110 tons per sq. ft.) for 4-inch to 16-inch cubes of concrete, 46 months old, composed of 1 part Rosendale cement paste, 1½ parts sand, and 6 parts broken stone. Under the same conditions, concrete composed of 1 Rosendale cement paste, 3 sand, and 6 broken stone stood 1,021 lbs. per sq. in. (73 tons per sq. ft.). Another sample of cement gave 1,078 lbs. per sq. in. (77 tons per sq. ft.) for concrete, 22 months

† Eliot C. Clarke, in Trans. Am. Soc. of C. E., vol. xiv. p. 166.

‡ Engineer's Pocket-book, p. 679.

§ History St. Louis Bridge, p. 328.

‖ Report for 1884,—Senate Ex. Doc. No. 85, 49th Congress, 1st Session,—pp. 166, 190, 198.

old, composed of 1 part cement paste, 3 sand, 2 gravel, and 4 broken stone. Ten experiments with a single sample of Portland cement gave 3,067 lbs. per sq. in. (219 tons per sq. ft.) for concrete composed of 1 part cement paste, 3 sand, and 6 broken stone. The concrete under the Washington Monument, composed of 1 Portland, 2 sand, 3 pebbles, and 4 broken stone, when 6 months old stood 2,000 lbs. per sq. in. (144 tons per sq. ft.).

Experiments made in connection with the construction of the Vyrnwy dam—built to impound water for the supply of Liverpool, England—gave an average strength from six experiments, for cubes of 1 to 2 Portland cement mortar* from 32 to 37 months old, crushed between pine cushions ½ inch thick, of 4,428 lbs. per sq. in. (284.7 tons per sq. ft.); and cubes of concrete composed of gravel and sufficient 1 to 2 Portland cement mortar to fill the interstices gave an average strength, for two cubes 35 and 36 months old, of 3,497 lbs. per sq. in. (224.9 tons per sq. ft.), and for two cubes 12 and 15 months old, of 4,583 lbs. per sq. in. (294.7 lbs. per sq. ft.). The blocks were made from the concrete actually used in the work, and were moulded by ordinary workmen without supervision, with the intention of securing blocks representative of the concrete as laid in the work. For cubes of the concrete tested between "mill-boards" (straw-boards), the same series of experiments gave results as follows: †

Age of the Blocks, months.	No. of Experiments.	Mean Crushing Strength, lbs. per sq. in.	Mean Crushing Strength, tons per sq. ft.
32—36	3	2,365	170.4
20—30	6	2,278	164.0
5—8	2	1,742	125.5
1—2½	7	1,477	106.4

158. Cost. The cost of concrete varies greatly, depending upon the kind of mortar—whether lime or cement,—upon the richness of the mortar, upon the proportion of ballast to mortar, upon the cost of the ingredients and of the labor, etc.

The following is the analysis of the composition and cost of the concrete employed for laying the foundations of the sea-wall at Lovell's Island, Boston Harbor: ‡

* Tensile strength tested neat, after being 6 days in air and 7 days in water = 638 lbs. per sq. in.

† Compiled from Deacon's Report on the Vyrnwy Masonry Dam.

‡ Compiled from Gillmore's Limes, Hydraulic Cements, and Mortars, p. 247.

Cement	0.83 bbl.	@ $1 54 =	$1 26
Sand	0.25 cu. yd.	@ 70 =	17
Gravel	0.90 cu. yd.	@ 27 =	24
Total materials	1.27 cu. yds.	=	$1 67
Labor, making mortar	0.06 day	@ $1 20 =	$0 08
" making concrete	0.11 day	@ 1 20 =	13
" transporting concrete	0.06 day	@ 1 20 =	08
" packing concrete	0.03 day	@ 1 20 =	04
Total labor	0.26 day	=	$0 33
Tools, implements, etc.			$0 11
Total cost 1 cu. yd. of concrete, in place			2 11

The proportions for this concrete were 1 cement, 3 sand, and 4 gravel. It was unusually cheap, owing partly to the use of pebbles instead of broken stone. The latter would have cost probably 4 to 6 times as much as the gravel. The amount of labor required was also unusually small, this item alone being frequently 6 to 8 times as much as in this case.

The following is the analysis* of the cost of nearly 10,000 yards of concrete as laid for the foundations of a blast-furnace plant near Troy, N. Y., in 1886. The conditions were unusually favorable for cheap work.

Cement	1.28 bbls.	@ $1 00	=	$1 28
Sand	0.10 cu. yd.	@ 0 80	=	08
Gravel	0.36 cu. yd.	@ 0 30	=	11
Broken Stone	0.74 cu. yd.	@ 1 41	=	1 04
Total materials	1.28 cu. yds.		=	$2 41
Labor, handling cement	0.02 day	@ $1 00	=	$0 02
" unloading stone	0.14 day	@ 1 00	=	14
" mixing	0.85 day	@ 1 00	=	85
" superintendence	0.01 day	@ 9 61	=	10
Total labor	1.02 days		=	$1 11
Total cost of a cubic yard of concrete, in place				$3 52

* Trans. Am. Soc. of C. E., vol. xv. p. 875.

The contract price for concrete in place in bridge piers and abutments at Chicago in 1887 was as follows:

1 part Portland, 3 sand, 6 broken stone,—$9.00 per cubic yard;
1 part Rosendale, 3 sand, 5 broken stone,—$6.00 per cubic yard.

The following is the cost, per cubic yard in place, of the concrete used in the construction of Hiland Avenue reservoir, Pittsburgh, Penn.* The stone was broken so as to pass through a 2½-inch ring. The mortar was 1 part Rosendale cement to 2 parts sand. The concrete was 1 part mortar to 2½ of stone. The concrete was mixed by hand. Common laborers received $1.25 per day, and foremen $2.50. The contract price was $6.00 per yard.

Quarrying stone	$0 45
Transporting stone	50
Breaking stone	85
Cement @ $1.35 per bbl	1 80
Sand, cost of digging	10
Water	05
Labor, mixing and laying	75
Incidentals	05
Total cost per cubic yard, in place	$4 05

For additional data concerning the cost of concrete, see §§ 233–34.

Art. 3. Artificial Stone.

159. Several kinds of artificial stone have come into use within the last twenty-five years for architectural and artistic purposes, and for the pavements of cellars, for footpaths, areas, and other localities not subjected to the tread of heavy animals. They are all a combination of hydraulic cement, sand, pebbles, etc. Some of them possess very considerable positive merit, and are of great value in districts where durable and cheap building-stone is not supplied by nature.

The strength and hardness of all varieties of artificial stone vary directly with the ultimate strength and hardness attainable by the hydraulic ingredients employed. An obvious means of improving their quality, therefore, is the employment of the highest grades of cement.

* Emile Low in *Engineering News*, vol. xiii. pp. 51-2.

160. Béton-Coignet. As made by its inventor, Coignet, of Paris, its usual ingredients are: Portland cement, siliceous hydraulic lime (like that obtained at Teil, France), and clean sand, mixed together with a little fresh water. The proportions are varied considerably for different kinds of work. The dry ingredients are first moistened very slightly with clean water, then thoroughly mixed by hand, and again mixed in a mill. Moulds are then filled with the mixture, which is compacted by ramming.

The peculiarities of this stone result from the small quantity of water used in its manufacture and the thoroughness with which the mixing is done. It is nothing more than hydraulic concrete, from which the coarse fragments have been omitted, and upon which have been conferred all the advantages to be derived from a judicious choice in the quality and proportions of the ingredients, and from their thorough manipulation. It sets quickly, is very strong, and is the best béton or concrete. It may be made into blocks to be used as cut stone, or it may be built up into immense masses of any desired shape by moulding the different parts in place. It is used to a considerable extent in constructing the walls of houses, and in repairing masonry,*—as bridge piers, culverts, etc. It is used very extensively in France.

In this country a mixture of Rosendale cement and sand is frequently called Béton-Coignet.

161. Portland Stone. This is a mixture of Portland cement and sand, or sand and gravel, compacted into form by tamping. When properly made, it possesses the essential requisites of strength and hardness in a degree proportionate to the value of the cement employed. The proportions of 1 measure of dry cement to 2 or $2\frac{1}{2}$ measures of sand will answer for most purposes. The manipulation should be prolonged and thorough to insure the production of a homogeneous stone. It is much used for flagging, for which purpose the surface layer, to the thickness of about half an inch, may advantageously be composed of 1 measure of cement to $1\frac{1}{4}$ or $1\frac{1}{2}$ of sand, and the coarse gravel should be omitted.

162. McMurtrie Stone. This stone, the process for making which is patented, consists essentially of the Portland stone described above, in the pores of which are formed compounds of

* Trans. Am. Soc. of C. E., vol. x. pp. 291–308.

alumina with the fatty acids by the double decomposition of alum and a potash soap (see § 141, page 96). These compounds are insoluble in water, are not acted upon by the carbonic acid of the air, and add considerably to the early strength of the stone and somewhat to its ultimate strength.

The peculiar merit of this stone is that its power of absorbing water is decreased by the use of the alum and the soap. All mortars and most of the artificial stones absorb water freely,—porous mortar from 50 to 60 per cent. of its own weight and the best Portland from 10 to 20 per cent.,—and consequently they disintegrate rapidly under the action of frost. The absorbed water also dissolves the salts of magnesia, lime, soda, and potash (of all of which there is always more or less in cement), and on evaporating leaves a white efflorescence on the surface, which injures the appearance of the wall. For these reasons the ordinary artificial stones are in disrepute for architectural purposes. The absorptive power of the McMurtrie stone is about twice that of granite, about equal to that of the best limestones, and about one tenth or less of that of the best sandstones. It has been used in Washington, D. C., to a limited extent, the window trimmings of the National Museum and also the fronts of a few stores and dwellings being of this stone. It appears to have given entire satisfaction.

163. Frear Stone. This is composed of siliceous sand and good Portland cement, to which gum shellac is added. The composition used by the inventor was 1 measure of cement and 2½ measures of sand moistened with an alkaline solution of shellac of sufficient strength to furnish an ounce of the shellac to a cubic foot of stone. The shellac adds to the early strength of the stone; but it is not certain that it adds to the ultimate strength, nor is it certain that the shellac may not decay and ultimately prove an element of weakness.

When mixed, it is rammed into wooden moulds, and after setting is laid away to season,—which requires several months for best results. It was much used in architectural work in the West a few years ago, but did not give satisfaction.

164. Ransome Stone. This is made by forming in the interstices of sand, gravel, or any pulverized stone a hard and insoluble cementing substance, by the natural decomposition of two chemical compounds in solution. Sand and the silicate of

soda are mixed in the proportion of a gallon of the latter to a bushel of the former and rammed into moulds, or it may be rolled into slabs for footpaths, etc. At this stage of the process the blocks or slabs may be easily cut into any desired form. They are then immersed, under pressure, in a hot solution of chloride of calcium, after which they are thoroughly drenched with cold water —for a longer or shorter period, according to their size—to wash out the chloride of sodium formed during the operation. In England grindstones are frequently made by this process.

165. SOREL STONE. Some years ago, M. Sorel, a French chemist, discovered that the oxychloride of magnesium possessed hydraulic energy in a remarkable degree. This cement is the basis of the Sorel stone. It is formed by adding a solution of chloride of magnesium, of the proper strength and in the proper proportions, to the oxide of magnesium. The strength of this stone, as well as its hardness, exceeds that of any other artificial stone yet produced, and may, when desirable, be made equal to that of the natural stone which furnishes the powder or sand used in its fabrication. The process is patented, and is used mainly in making emery-wheels. By incorporating large pebbles and cobble-stones in the mixture the stone can be made quite cheaply, and is therefore suitable for foundations and plain massive walls.

CHAPTER V.

QUARRYING.

166. THIS is so large a subject that it cannot be more than entered upon here; for greater detail, see treatises on Quarrying, Rock-blasting, and Tunneling.

167. SOURCES OF BUILDING STONES. The bowlders, which are scattered promiscuously over the surface of the ground and also frequently buried in it, furnish an excellent building stone for massive structures where strength is essential. They are usually of tough granite or of a slaty structure, and are difficult to work. Sometimes they have a cleavage plane or rift, along which they may be split. They can be broken into irregular pieces by building a fire about them, and drenching them while hot with water, or they may be broken by explosives.

Of course by far the greater quantity of stone is taken directly from quarries. All building-stone deposits have usually a certain amount of covering, consisting either of a portion of the same deposit, which has been disintegrated by atmospheric influences, or of a later deposit. This covering is called the "cap-rock" or "stripping." In opening the quarry, the solid portions of cap-rock are broken up by blasting, and the whole is carted out of the way. After a sufficient space is stripped, the next step necessary, when the quarry rock does not stand out in cliffs, is to excavate a narrow space on one side for a quarry face, either by blasting or by some of the methods to be described presently.

168. METHODS OF QUARRYING. After a considerable area has thus been laid bare, the stone is quarried in one of three ways.

169. *1.* **By Hand Tools.** When the stone is thin-bedded, it may be quarried by hand-tools alone. The principal tools are pick, crow-bar, drill, hammer, wedge, and plug and feathers. The layers are forced apart by the crow-bar or wedges. The flat pieces are broken up with the hammer or by drilling holes for the plug and feathers.

The plug is a narrow wedge with plane faces; the feathers are wedges flat on one side and rounded on the other (see Fig. 25). When a plug is placed between two feathers, the three will slip into a cylindrical hole; if the plug is then driven, it exerts a great force. If these plugs and feathers are placed a few inches apart in a row, and all driven at the same time, the stone will be cracked along the line of the holes, even though it be comparatively thick.

The drill used to cut the holes for the plug and feathers is a bar of steel furnished with a wide edge sharpened to a blunt angle and hardened. It is operated by one man, who holds the drill with one hand and drives it with a hammer in the other, rotating the drill between blows. The holes are usually from $\frac{3}{8}$ to $\frac{3}{4}$ of an inch in diameter.

Sandstones and limestones occurring in layers thin enough to be quarried as above are usually of inferior quality, suitable only for slope walls, paving, riprap, concrete, etc.

170. *II.* **By Explosives.** Generally, the cheapest method of quarrying small blocks is by the use of explosives. However, explosives are used mainly for detaching large blocks, which are afterwards worked up by means of wedges. In this method of quarrying, drill-holes are put down to the depth to which the rock is to be split, and the requisite amount of powder or other explosive put in, covered with sand, and fired by a fuse. Sometimes numerous charges in a line of drill-holes are fired simultaneously by means of electricity.

Quick-acting explosives, like dynamite, have a tendency to shatter the stone and break it in many directions, the texture being affected by the sudden explosion in the same manner as by the blow of a hammer. Coarse gunpowder is generally preferred for quarrying stone. Light charges of powder lightly covered with sand are better than heavy charges tightly tamped; * and experience goes to show that better work is done by repeated light blasts in the same hole, than by a single heavy blast. By means of light charges often repeated, a mass of rock may be detached without being broken up, which would be badly shattered by a single charge strong enough to detach it.

In each locality the structure of the rock must be carefully

* For an article showing that an air-space should be left between the explosive and the tamping, see *Engineering News*, vol. xviii. p. 382.

studied with a view to take advantage of the cleavage planes and natural joints. For quarrying each class of rocks there is a characteristic method employed, which is, however, varied in detail in different quarries. The minor details of quarry methods are as various as the differences existing in the textures, structures, and modes of occurrence of the rocks quarried. Much depends upon how the blast is made. The direction in which the rock is most liable to break depends upon the structure of the rock and the shape of the drill-hole. Even such an apparently unimportant matter as the form of the bottom of the drill-hole into which the explosive is put has a very marked effect. If bored with a hand-drill, the hole is generally triangular at the bottom, and a blast in such a hole will break the rock in three directions. In some quarries the lines of fracture are made to follow predetermined directions by putting the charge of powder into canisters of special forms.*

171. *Drills.* The holes are bored by jumpers, churn-drills, or machine-drills. The first is a drill similar to the one used for drilling holes for plugs and feathers (§ 169), except that it is larger and longer. It is usually held by one man, who rotates it between the alternating blows from hammers in the hands of two other men. Churn-drills are long, heavy drills, usually 6 to 8 feet in length. They are raised by the workmen, let fall, caught on the rebound, raised and rotated a little, and then dropped again, thus cutting a hole without being driven by the hammer. They are more economical than jumpers, especially for deep holes, as they cut faster and make larger holes than hand-drills.

172. Machine rock-drills bore much more rapidly than hand-drills, and also more economically, provided the work is of sufficient magnitude to justify the preliminary outlay. They drill in any direction, and can often be used in boring holes so located that they could not be bored by hand. They are worked either by steam directly, or by air compressed by steam or water-power and stored in a tank called a receiver and thence led to the drills through iron pipes.

A variety of rock-drilling machines has been invented,† but they can be grouped in two classes, viz., percussion-drills and rotating drills. The method of action of the percussion-drill is the same

* See Report on Quarry Industry in Vol. X. of the 10th Census, pp. 33, 34.

† For a full account of the more important ones, see Drinker's "Tunneling."

as that of the churn-drill already described. The usual form is that of a cylinder, in which a piston is moved by steam or compressed air, and the drill is attached to this piston so as to make a stroke with every complete movement of the piston. An automatic device causes it to rotate slightly at each stroke.

173. In the rotating drills, the drill-rod is a long tube, revolving about its axis. The end of the tube—hardened so as to form an annular cutting edge—is kept in contact with the rock, and by its rotation cuts in it a cylindrical hole, generally with a solid core in the center. The drill-rod is fed forward, or into the hole, as the drilling proceeds. The *débris* is removed from the hole by a constant stream of water which is forced to the bottom of the hole through the hollow drill-rod, and which carries the *débris* up through the narrow space between the outside of the drill-rod and the sides of the hole.

The diamond drill is the only form of rotary rock-drill extensively used in this country. The tube has a head at its lower end, in which are set a number of carbons or black diamonds. The diamonds usually project slightly beyond the circumference of the head, which is perforated to permit the ingress and egress of the water used in removing the *débris* from the hole and at the same time prevent the head from binding in the hole. When it is desirable to know the precise nature and stratification of the rock penetrated, the cutting points are so arranged as to cut an annular groove in the rock, leaving a solid core, which is broken off and lifted out whenever the head is brought up. Where it is not desired to preserve the core intact, a solid boring-bit is used instead of the core-bit. They are made of any size up to 15 inches in diameter.

174. *Explosives.** The principal explosives are gunpowder, nitro-glycerine, and dynamite. Only a coarse-grained and cheap variety of the first is used in quarrying, the others being too sudden and too strong in their action.

The pressure exerted by gunpowder when fired in a confined space depends upon the relative weight and quality of powder used, and upon the space occupied by the gases evolved. The absolute force of gunpowder, the force which it exerts when it exactly fills the space in which it is confined, has never been satisfactorily ascer-

* For a full account of all the various explosives, see Drinker's "Tunneling;" and Drinker's "Modern Explosives."

tained. It has been variously estimated at from 15,000 to 1,500,000 pounds per square inch. Experiments by Gen. Rodman show that for the powder used in gunnery the absolute force of explosion is at least 200,000 pounds per square inch. "In ordinary quarrying, a cubic yard of solid rock in place (or about 1.9 cubic yards piled up after being quarried) requires from ¼ to ¾ pound of powder. In very refractory rock, lying badly for quarrying, a solid yard may require from 1 to 2 pounds. In some of the most successful great blasts for the Holyhead Breakwater, Wales, (where several thousands of pounds of powder were exploded, usually by galvanism, at a single shot,) from 2 to 4 cubic yards (solid) were *loosened* per pound of powder; but in many instances not more than 1 to 1½ yards. Tunnels and shafts require 2 to 6 pounds per solid yard, usually 3 to 5 pounds. Soft, partially decomposed rock frequently requires more than harder ones." *

The explosion of the powder splits and loosens a mass of rock whose volume is approximately proportional to the *cube of the line of least resistance*,—that is, of the shortest distance from the charge to the surface of the rock,—and may be roughly estimated at *twice* that cube; but this proportion varies much in different cases. The ordinary rule for the weight of powder in small blasts is

$$\text{POWDER, } \textit{in pounds}, = (\text{LINE OF RESISTANCE, } \textit{in feet},)^2 \div 32.$$

Powder is sold in kegs of 25 lbs., costing about $2.00 to $2.25 per keg, exclusive of freight,—which is very high, owing to the risk.

175. Most of the explosives which of late years have been taking the place of gunpowder consist of a powdered substance, partly saturated with nitro-glycerine—a fluid produced by mixing glycerine with nitric and sulphuric acids. Nitro-glycerine, and the powders containing it, are always exploded by means of sharp percussion, which is applied by means of a cap and fuse. The cap is a hollow copper cylinder, about ¼ inch in diameter and an inch or two in length, containing a cement composed of fulminate of mercury and some inert substance. The cap is called single-force, double-force, etc., according to the amount of explosive it contains.

The principal advantages of nitro-glycerine as an explosive consist (1) in its instantaneous development of force, due to the fact that, pound for pound, it produces at least three and a half times

* Trautwine's Engineer's Pocket-book.

as much gas, and twice as much heat, as gunpowder; and (2) in its high specific gravity, which permits the use of small drill-holes.

Nitro-glycerine is rarely used in the liquid state in ordinary quarrying or blasting, owing to the liability of explosion through accidental percussion, and owing to its liability to leakage. It explodes so suddenly that very little tamping is required, the mere weight of moist sand, earth, or water being sufficient. This fact, and the additional one that nitro-glycerine is unaffected by immersion in water and is heavier than water, render it particularly suitable for sub-aqueous work, or for holes containing water. If the rock is seamy, the nitro-glycerine must be confined in water-tight casings. Such casings, however, necessarily leave some spaces between the rock and the explosive, which diminishes the effect of the latter. The liquid condition of nitro-glycerine is useful in causing it to fill the drill-hole completely, so that there are no empty spaces in it to waste the force of the explosion. On the other hand, the liquid form is a disadvantage, because when thus used in seamy rock without a containing vessel portions of the nitro-glycerine leak away and remain unexploded and unsuspected, and may cause accidental explosion at a future time.

The price of nitro-glycerine is from 50 to 60 cents per quart.

176. Dynamite is the name given to any explosive which contains nitro-glycerine mixed with a granular absorbent. If the absorbent is inert, the mixture is called *true dynamite;* if the absorbent itself contains explosive substances, the mixture is called *false dynamite.* The absorbent, by its granular and compressible condition, acts as a cushion to the nitro-glycerine, and protects it from percussion and from the consequent danger of explosion, but does not diminish its power when exploded. Nitro-glycerine undergoes no change in composition by being absorbed; and it then freezes, burns, explodes, etc., under the same conditions as to pressure, temperature, etc., as when in the liquid form. The cushioning effect of the absorbent merely renders it more difficult to bring about sufficient percussive pressure to cause explosion. The absorption of the nitro-glycerine in dynamite renders the latter available in horizontal holes or in holes drilled upward. True dynamite loses only a very small percentage of its explosive power when saturated with water, but is then much more difficult to explode.

True dynamites must contain at least 50 per cent. of nitro-glycerine, otherwise the latter will be too completely cushioned by the absorbent, and the powder will be too difficult to explode. False dynamites, on the contrary, may contain as small a percentage of nitro-glycerine as may be desired, some containing as little as 15 per cent. The added explosive substances in the false dynamites generally contain large quantities of oxygen, which are liberated upon explosion, and aid in effecting the complete combustion of any noxious gases arising from the nitro-glycerine. The false are generally inferior to the true dynamites, since the bulk of the former is increased in a higher ratio than the power; and as the cost of the work is largely dependent upon the size of the drill-holes, there is no economic gain.

Dynamites which contain large percentages of nitro-glycerine explode with great suddenness, tending to break the rock into small fragments. They are most useful in blasting very hard rock. In such rock dynamite containing 75 per cent. of nitro-glycerine is roughly estimated to have about 6 times the force of an equal weight of gunpowder; but in soft rock or clay its power, at equal cost, is inferior to that of common gunpowder, because its action is akin to a sudden blow, rather than to a continued push. For soft or decomposed rocks, sand, and earth, the lower grades of dynamite, or those containing a smaller percentage of nitro-glycerine, are more suitable. They explode with less suddenness, and their tendency is rather to upheave large masses of rock than to splinter small masses.

"Judgment must be exercised as to the grade and quantity of explosive to be used in any given case. Where it is not objectionable to break the rock into small pieces, or where it is desired to do so for convenience of removal, the higher shattering grades are useful. Where it is desired to get the rock out in large masses, as in quarrying, the lower grades are preferable. For very difficult work in hard rock, and for submarine blasting, the highest grades, containing 70 to 75 per cent. of nitro-glycerine, are used. A small charge does the same execution as a larger charge of lower grade, and of course does not require the drilling of so large a hole. In submarine work their sharp explosion is not deadened by the water. For general railroad work, ordinary tunneling, mining of ores, etc., the average grade, containing 40 per cent. of nitro-glycer-

ine, is used ; for quarrying, 35 per cent.; for blasting stumps, trees, piles, etc., 30 per cent.; for sand and earth, 15 per cent."

177. A great variety of dynamites is made. Each manufacturer usually makes a number of grades, containing different percentages of nitro-glycerine, and gives to his powder some fanciful name. Dynamite is sold in cylindrical, paper-covered cartridges, from $\frac{7}{8}$ of an inch to 2 inches in diameter, and 6 to 8 inches long, or longer, which are packed in boxes containing 25 or 50 pounds each. They are furnished, to order, of any required size. The price per pound ranges from 15 cents for 15 per cent. nitro-glycerine to 50 cents for 75 per cent. nitro-glycerine.

Table 14 (page 124) gives the names of all the explosives containing nitro-glycerine, with the per cent. in each case.*

178. *III.* By Channeling and Wedging. By channeling is meant the process of cutting long narrow channels in rock to free the sides of large blocks of stone. Quite a large number of machines have been invented for doing this work, all of which make the channels by one form or the other of the machine drills already described (see the second paragraph of § 172). The machines are mounted upon a track on the bed of the quarry, and can be moved forward as the work progresses. If the rock is in layers, it is only necessary to cut the channels part way through the layer, when the block can be detached with wedges, the groove guiding the fracture. If the rock is not in layers, after the necessary channels have been cut around the block, it is necessary to under-cut the block in order to release it. This is accomplished by drilling a series of holes along the bottom, which process is called "gadding" by quarry-men. The block is then split from its bed by means of wedges. The method of channeling and wedging is much employed in quarrying marble, the massive limestones, and the thick-bedded sandstones. The method is very economical and expeditious, except in granite and the hardest sandstones. For illustrations of the two principal channeling machines and also quarries being worked by this method, see Report on the Quarry Industry, pp. 44–52, in Vol. X. of the Tenth Census of the United States.

* W. C. Foster, in *Engineering News*, vol. xix. p. 254. For a list of all the explosives employed as blasting agents, together with a description of their composition and references to the literature of each, see *Engineering News*, vol. xix. pp. 533–34, and vol. xx. pp. 8–10.

TABLE 14.

List of Explosives containing Nitro-glycerine.

Name of Explosive.	Per cent. of Nitro-glycerine.	Name of Explosive.	Per cent. of Nitro-glycerine.
Ætna powder, No. 1	65	Glyxoline	....
" " " 2XX	50	Hecla powder, No. 1XX	75
" " " 2	40	Gun Sawdust	16 to 20
" " " 3X	35	" " No. 1X	50
" " " 4X	25	" " " 1	40
" " " 5	15	" " " 2X	35
Ammonia powder	16 to 20	" " " 2	30
Asbestos powder	varies	" " " 3X	25
Atlas powder, A	75	" " " 3	20
" " B+	60	Hercules powder, No. 1XX	75
" " B	50	" " " 1	65
" " C+	45	" " " 2SSS	55
" " C	40	" " " 2SS	50
" " D+	35	" " " 2S	45
" " D	30	" " " 2	40
" " E+	25	" " " 3S	35
" " E	20	" " " 3	30
" " F+	15	" " " 4S	25
Brady's dynamite	33	" " " 4	20
Brain's powder	40	Horsley's powder (some	
Colonia powder	40	varieties)	20
Dualin (Dittmar's)	50	Judson Giant Powder, No. 2	40
Dynamite (Nobel's, Kiesel-		Judson powder, FFF	20
guhr dynamite),		FF	15
Old No. 1	75	F	10
Old No. 2	40	RRP	5 to 6
Old No. 3	25	Lithofracteur	52
Electric powder	33	Metalline Nitroleum	varies
Explosive gelatine	93	Mica powder, No. 1	40
Forcite, 2 grades	75, 70	" " " 2	52
Fulgurite (solid)	60	Miners' Powder Co.'s Dy-	
" (liquid)	90	namite	33
Gelatine dynamite, A	97.5	Neptune powder	32.7
" " No. 1	58	Nitro Tolnol	70
" " " 2	33.8	Norrbin & Ohlsson's pow-	
Gelatine explosive de		der	25 to 50
guerre	89.3	Pantopolite	....
Gelignite	56.5	Porifera Metroleum	varies
Giant powder, No. 1	75	Rendrock	33.4
" " New " 1	50	Sebastin, No. 1	78
" " " 2 extra	45	" " 2	68
" " " 2	40	Selenitic powder	varies
" " " 2c	33	Seranim	....
" " " XXX	27	Vigorite (U. S.)	43.8
" " " M	20	Vitrite, No. 1	....
Giant powder (Nobel's),		" " 2	....
No. 2	20	Vulcan powder	32.6

CHAPTER VI.

STONE CUTTING.

ART. 1. TOOLS.

179. IN order to describe intelligibly the various methods of preparing stones for use in masonry, it will be necessary to begin with a description of the tools used in stone-cutting, as the names of many kinds of dressed stones are directly derived from those of the tools used in dressing them.

With a view to securing uniformity in the nomenclature of building stones and of stone masonry, a committee of the American Society of Civil Engineers prepared a classification and recommended that all specifications should be made in accordance therewith. The old nomenclature was very unsystematic and objectionable on many grounds. The new system is good in itself, is recommended by the most eminent authority, has been quite generally adopted by engineers, and should therefore be strictly adhered to. The following description of the *hand tools* used in stone cutting is from the report of the American Society's committee.*

180. HAND TOOLS. "The *Double Face Hammer*, Fig. 9, is a heavy tool weighing from 20 to 30 pounds, used for roughly shaping stones as they come from the quarry and for knocking off projections. This is used only for the roughest work.

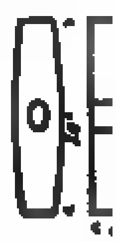

FIG. 9.—DOUBLE FACE HAMMER.

"The *Face Hammer*, Fig. 10, has one blunt and one cutting end, and is used for the same purpose as the double face hammer where less weight is required. The cutting end is used for roughly squaring stones, preparatory to the use of finer tools.

FIG. 10.—FACE HAMMER.

* Trans. Am. Soc. of C. E., vol. vi. pp. 297–304.

"The *Cavil*, Fig. 11, has one blunt and one pyramidal, or pointed, end, and weighs from 15 to 20 pounds. It is used in quarries for roughly shaping stone for transportation.

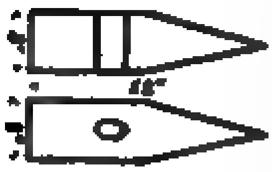

FIG. 11.—CAVIL.

The *Pick*, Fig. 12, somewhat resembles the pick used in digging, and is used for rough dressing, mostly on limestone and sandstone. Its length varies from 15 to 24 inches, the thickness at the eye being about 2 inches.

FIG. 12.—PICK.

"The *Ax*, or *Pean Hammer*, Fig. 13, has two opposite cutting edges. It is used for making drafts around the arris, or edge, of stones, and in reducing faces, and sometimes joints, to a level. Its length is about 10 inches, and the cutting edge about 4 inches. It is used after the point and before the patent hammer.

FIG. 13.—AX.

"The *Tooth Ax*, Fig. 14, is like the ax, except that its cutting edges are divided into teeth, the number of which varies with the kind of work required. This tool is not used in granite and gneiss cutting.

FIG. 14.—TOOTH AX.

"The *Bush Hammer*, Fig. 15, is a square prism of steel whose ends are cut into a number of pyramidal points. The length of the hammer is from 4 to 8 inches, and the cutting face from 2 to 4 inches square. The points vary in number and in size with the work to be done. One end is sometimes made with a cutting edge like that of the ax.

FIG. 15.—BUSH HAMMER.

"The *Crandall*, Fig. 16, is a malleable-iron bar about two feet

long, slightly flattened at one end. In this end is a slot 3 inches long and $\frac{5}{8}$ inch wide. Through this slot are passed ten double-headed points of $\frac{1}{4}$-inch square steel, 9 inches long, which are held in place by a key.

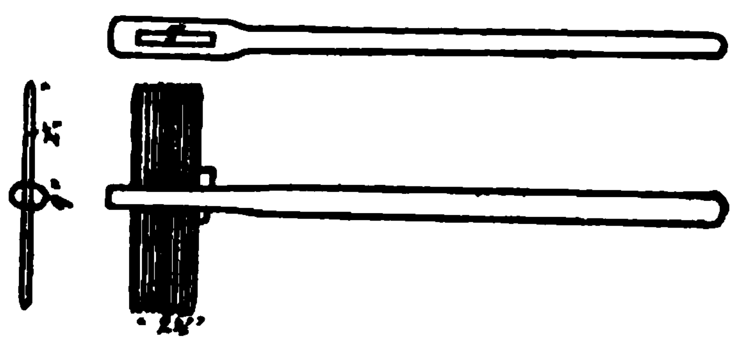

FIG. 16.—CRANDALL.

"The *Patent Hammer*, Fig. 17, is a double-headed tool so formed as to hold at each end a set of wide thin chisels. The tool is in two parts, which are held together by the bolts which hold the chisels. Lateral motion is prevented by four guards on one of the pieces. The tool without the teeth is $5\frac{1}{2} \times 2\frac{3}{4} \times 1\frac{1}{2}$ inches. The teeth are $2\frac{3}{4}$ inches wide. Their thickness varies from $\frac{1}{12}$ to $\frac{1}{8}$ of an inch. This tool is used for giving a finish to the surface of stones.

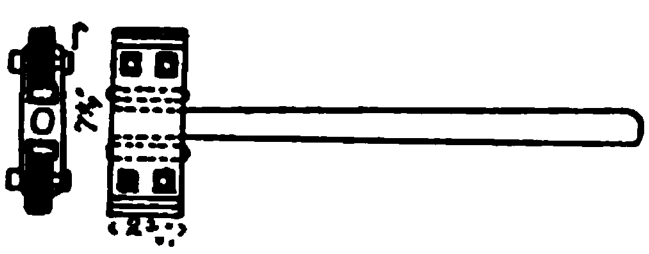

FIG. 17.—PATENT HAMMER.

"The *Hand Hammer*, Fig. 18, weighing from 2 to 5 pounds, is used in drilling holes, and in pointing and chiseling the harder rocks.

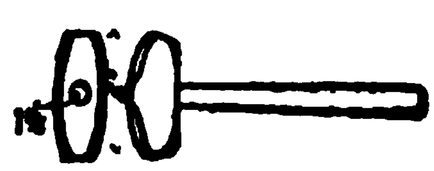

FIG. 18.—HAND HAMMER.

"The *Mallet*, Fig. 19, is used where the softer limestones and sandstones are to be cut.

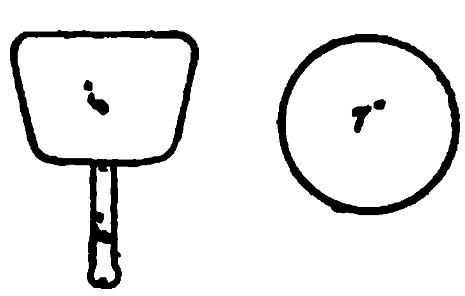

FIG. 19.—MALLET.

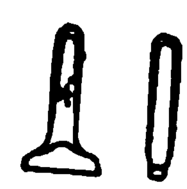

FIG. 20. PITCHING CHISEL.

"The *Pitching Chisel*, Fig. 20, is usually of $1\frac{1}{4}$-inch octagonal steel, spread on the cutting edge to a rectangle of $\frac{1}{8} \times 2\frac{1}{2}$ inches. It is used to make a well-defined edge to the face of a stone, a line being marked on the joint surface to which the chisel is applied and the portion of the stone outside of the line broken off by a blow with the hand-hammer on the head of the chisel.

"The *Point*, Fig. 21, is made of round or octagonal rods of steel, from $\frac{1}{4}$ inch to 1 inch in diameter. It is made about 12 inches long, with one end brought to a point. It is used until its length is reduced to about 5 inches. It is employed for dressing off the irregular surface of stones, either for a permanent finish or preparatory to the use of the ax. According to the hardness of the stone, either the hand-hammer or the mallet is used with it.

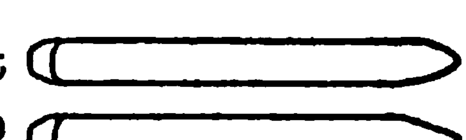

FIG. 21.—POINT.

"The *Chisel*, Fig. 22, of round steel of $\frac{1}{2}$ to $\frac{3}{4}$ inch in diameter and about 10 inches long, with one end brought to a cutting edge from $\frac{1}{2}$ inch to 2 inches wide, is used for cutting drafts or margins on the face of stones.

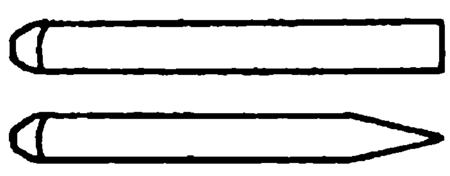

FIG. 22.—CHISEL.

"The *Tooth Chisel*, Fig. 23, is the same as the chisel, except that the cutting edge is divided into teeth. It is used only on marbles and sandstones.

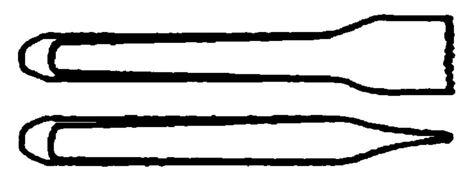

FIG. 23. TOOTH CHISEL.

FIG. 24. SPLITTING CHISEL.

"The *Splitting Chisel*, Fig. 24, is used chiefly on the softer, stratified stones, and sometimes on fine architectural carvings in granite.

"The *Plug*, a truncated wedge of steel, and the *Feathers* of half-round malleable iron, Fig. 25, are used for splitting unstratified stone. A row of holes is made with the *Drill*, Fig. 26, on the

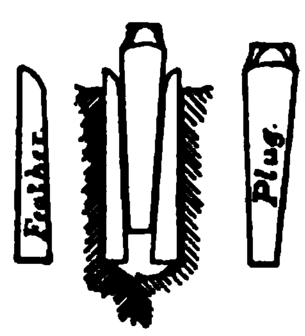

FIG. 25. PLUGS AND FEATHERS.

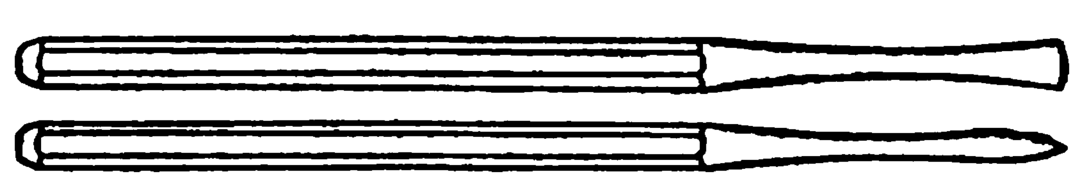

FIG. 26.—DRILLS.

line on which the fracture is to be made; in each of these holes two feathers are inserted, and the plugs lightly driven in between them. The plugs are then gradually driven home by light blows of the hand hammer on each in succession until the stone splits."

181. **MACHINE TOOLS.** In all large stone-yards machines are used to prepare the stone. There is great variety in their form, but since the surface never takes its name from the tool which forms it, it will be neither necessary nor profitable to attempt a description of individual machines. They include stone-saws, stone-cutters, stone-planers, stone-grinders, and stone-polishers.

The saws may be either drag, circular, or band saws; the cutting may be done by sand and water fed into the kerf, or by carbons or black diamonds. Several saws are often mounted side by side and operated by the same power.

The term "stone-cutter" is usually applied to the machine which

attacks the rough stone and reduces the inequalities somewhat. After this treatment the stone goes in succession to the stone-planer, stone-grinder, and stone-polisher.

Those stones which are homogeneous, strong and tough, and comparatively free from grit or hard spots, can be worked by machines which resemble those used for iron; but the harder, more brittle stones require a mode of attack more nearly resembling that employed in dressing stone by hand. Stone-cutters and stone-planers employing both forms of attack are made.

Stone-grinders and stone-polishers differ only in the degree of fineness of the surface produced. They are sometimes called "rubbing-machines." Essentially they consist of a large iron plate revolving in a horizontal plane, the stone being laid upon it and braced to prevent its sliding. The abradent is sand, which is abundantly supplied to the surface of the revolving disk. A small stream of water works the sand under the stone and also carries away the *débris*.

ART. 2. METHOD OF FORMING THE SURFACES.

182. It is important that the engineer should understand the methods employed by the stone-cutter in bringing stones to any required form. The surfaces most frequently required in stone cutting are plane, cylindrical, warped, helicoidal, conical, spherical, and sometimes irregular surfaces.

183. **PLANE SURFACES.** In squaring up a rough stone, the first thing the stone-cutter does is to draw a line, with iron ore or black lead, on the edges of the stone, to indicate as nearly as possible the required plane surface. Then with the hammer and the pitching-tool he pitches off all *débris* or waste material above the lines, thereby reducing the surface approximately to a plane. With a chisel he then cuts a draft around the edges of this surface, *i. e.*, he forms narrow plane surfaces along the edges of the stone. To tell when the drafts are in the same plane, he uses two straight-edges having parallel sides and equal widths. See Fig. 27. The projections

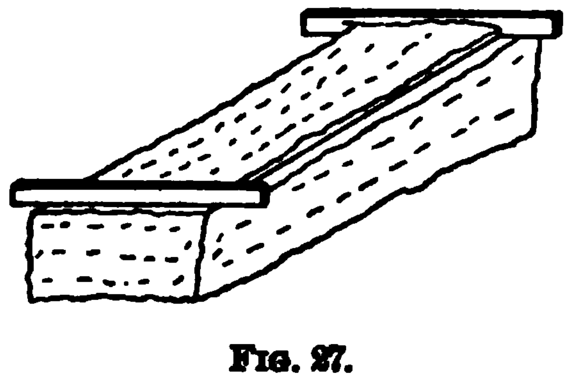

FIG. 27.

on the surface are then removed by the pitching chisel or the point, until the straight-edge will just touch the drafts and the intermediate surface when applied across the stone in any direction.

The surface is usually left a little "slack," *i. e.*, concave, to allow room for the mortar; however, the surface should be but a very little concave.

The surface is then finished with the ax, patent hammer, bush hammer, etc., according to the degree of smoothness required.

184. To form a second plane surface at right angles to the first one, the workman draws a line on the cut face to form the intersection of the two planes; he also draws a line on the ends of the stone approximately in the required plane. With the ax or the chisel he then cuts a draft at each end of the stone until a steel square fits the angle. He then joins these drafts by two others at right angles to them, and brings the whole surface to the same plane. The other faces may be formed in the same way.

If the surfaces are not at right angles to each other, a bevel is used instead of a square, the same general method being pursued.

185. CYLINDRICAL SURFACES. These may be either concave or convex. The former are frequently required, as in arches; and the latter sometimes, as in the outer end of the face-stones of an arch. The stone is first reduced to a paralellopipedon, after which the curved surface is produced in either of two ways: (1) by cutting a circular draft on the two ends and applying a straight-edge along the rectilinear elements (Fig. 28); or (2) by cutting a draft along the line of intersection of the plane and cylindrical surface, and applying a curved templet to the required surface (Fig. 29).

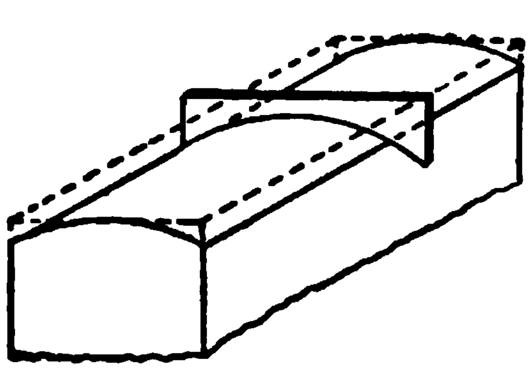

FIG. 28.

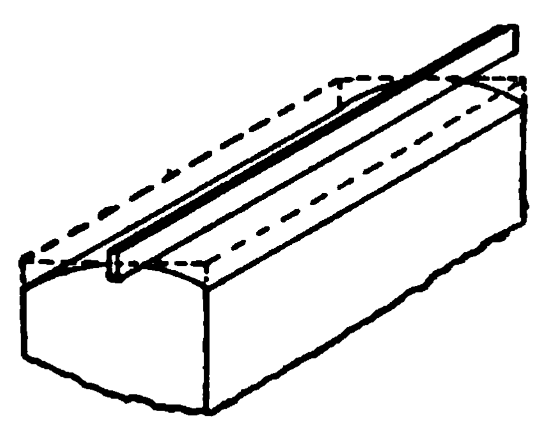

FIG. 29.

186. CONICAL SURFACES may be formed by a process very similar to the first one given above for cylindrical surfaces. Such surfaces are seldom used.

187. SPHERICAL SURFACES are sometimes employed, as in domes. They are formed by essentially the same general method as cylindrical surfaces.

188. WARPED SURFACES. Under this head are included what

the stone-cutters call "twisted surfaces," helicoidal surfaces, and the general warped surface. None of these are common in ordinary stone-work.

The method of forming a surface equally twisted right and left will be described; by obvious modifications the same method can be applied to secure other forms. Two twist rules are required, the angle between the upper and lower edges being half of the required twist. Drafts are then cut in the ends of the stone until the tops of the twist rules, when applied as in Fig. 30, are in a plane. The remainder of the projecting face is removed until a straight-edge, when applied parallel to the edge of the stone, will just touch the end drafts and the intermediate surface.

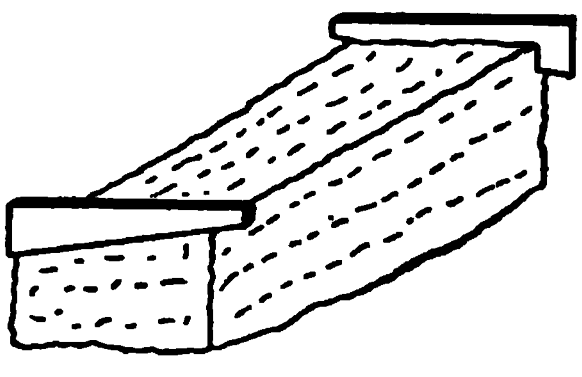

FIG. 30.

If the surface is to be twisted at only one end, a parallel rule and a twist rule are used.

189. MAKING THE DRAWINGS. The method of making working drawings for constructions in stone will appear in subsequent chapters, in connection with the study of the structures themselves; but for detailed instructions, see the text-books on Stereotomy or Stone Cutting.

ART. 3. METHODS OF FINISHING THE SURFACES.*

190. "All stones used in building are divided into three classes, according to the finish of the surface; viz.:

I. Rough stones that are used as they come from the quarry.

II. Stones roughly squared and dressed.

III. Stones accurately squared and finely dressed.

"In practice, the line of separation between them is not very distinctly marked, but one class gradually merges into the next.

191. I. "UNSQUARED STONES. This class covers all stones which are used as they come from the quarry, without other preparation than the removal of very acute angles and excessive projections from the general figure. The term 'backing,' which is frequently applied to this class of stone, is inappropriate, as it properly designates material used in a certain relative position in a wall, whereas stones of this kind may be used in any position.

192. II. "SQUARED STONES. This class covers all stones that

* This article is taken from the report of the committee of the American Society of Civil Engineers previously referred to.

are roughly squared and roughly dressed on beds and joints. The dressing is usually done with the face hammer or ax, or in soft stones with the tooth hammer. In gneiss it may sometimes be necessary to use the point. The distinction between this class and the third lies in the degree of closeness of the joints. Where the dressing on the joints is such that the distance between the general planes of the surfaces of adjoining stones is one half inch or more, the stones properly belong to this class.

"Three subdivisions of this class may be made, depending on the character of the face of the stones:

"(*a*) **Quarry-faced** stones are those whose faces are left untouched as they come from the quarry.

"(*b*) **Pitched-faced** stones are those on which the arris is clearly defined by a line beyond which the rock is cut away by the pitching chisel, so as to give edges that are approximately true.

"(*c*) **Drafted Stones** are those on which the face is surrounded by a chisel draft, the space inside the draft being left rough. Ordinarily, however, this is done only on stones in which the cutting of the joints is such as to exclude them from this class.

"In ordering stones of this class the specifications should always state the width of the bed and end joints which are expected, and also how far the surface of the face may project beyond the plane of the edge. In practice, the projection varies between 1 inch and 6 inches. It should also be specified whether or not the faces are to be drafted.

193. III. "CUT STONES. This class covers all squared stones with smoothly-dressed beds and joints. As a rule, all the edges of cut stones are drafted, and between the drafts the stone is smoothly dressed. The face, however, is often left rough where construction is massive.

"In architecture there are a great many ways in which the faces of cut stone may be dressed, but the following are those that will usually be met in engineering work:

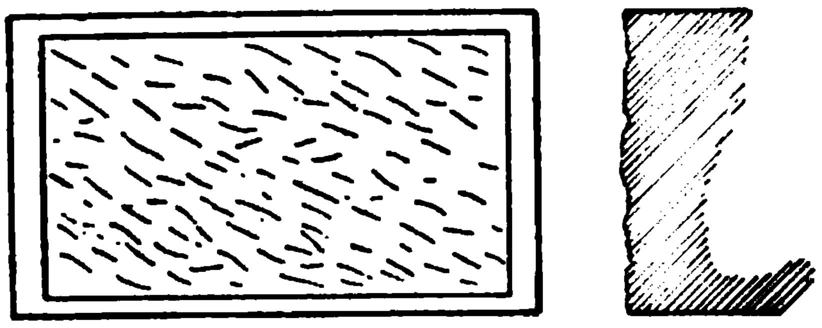

FIG. 81.—ROUGH-POINTED.

"**Rough-pointed.** When it is necessary to remove an inch or more from the face of a stone, it is done by the pick or heavy point until the projections

vary from $\frac{1}{2}$ inch to 1 inch. The stone is then said to be rough-pointed (Fig. 31). In dressing limestone and granite, this operation precedes all others.

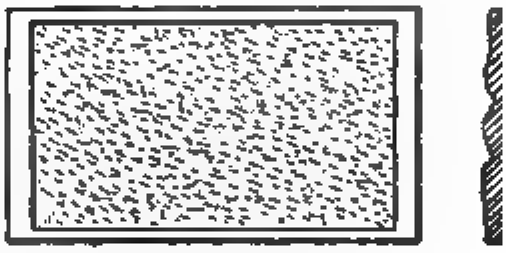

FIG. 32.—FINE-POINTED.

"**Fine-pointed.** (Fig. 32). If a smoother finish is desired, rough pointing is followed by fine pointing, which is done with a fine point. Fine pointing is used only where the finish made by it is to be final, and never as a preparation for a final finish by another tool.

"**Crandalled.** This is only a speedy method of pointing, the effect being the same as fine pointing, except that the dots on the stone are more regular. The variations of level are about $\frac{1}{8}$ inch, and the rows are made parallel. When other rows at right angles to the first are introduced, the stone is said to be *cross-crandalled.* Fig. 33.

FIG. 33.—CRANDALLED.

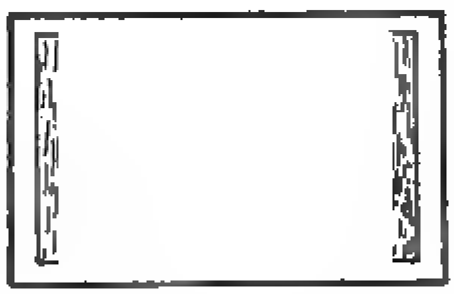

FIG. 34.—AXED.

"**Axed, or Pean-hammered, and Patent-hammered.** These two vary only in the degree of smoothness of the surface which is produced. The number of blades in a patent hammer varies from 6 to 12 to the inch; and in precise specifications the number of cuts to the inch must be stated, such as 6-cut, 8-cut, 10-cut, 12-cut. The effect of axing is to cover the surface with chisel marks, which are made parallel as far as practicable. Fig. 34. Axing is a final finish.

"**Tooth-axed.** The tooth-ax is practically a number of points, and it leaves the surface of a stone in the same condition as fine pointing. It is usually, however, only a preparation for bush-hammering, and the work is then done without regard to effect so long as the surface of the stone is sufficiently leveled.

"**Bush-hammered.** The roughnesses of a stone are pounded off by

the bush hammer, and the stone is then said to be 'bushed.' This kind of finish is dangerous on sandstone, as experience has shown that sandstone thus treated is very apt to scale. In dressing limestone which is to have a bush-hammered finish, the usual sequence of operation is (1) rough-pointing, (2) tooth-axing, and (3) bush-hammering. Fig. 35.

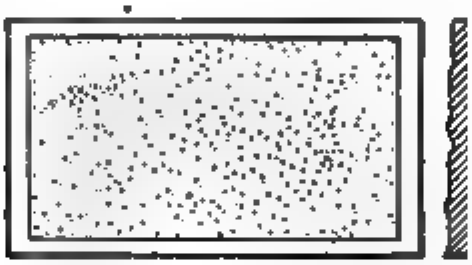

FIG. 35.—BUSH-HAMMERED.

"**Rubbed.** In dressing sandstone and marble, it is very common to give the stone a plane surface at once by the use of the stone-saw [§ 181]. Any roughnesses left by the saw are removed by rubbing with grit or sandstone [§ 181]. Such stones, therefore, have no margins. They are frequently used in architecture for string-courses, lintels, door-jambs, etc.; and they are also well adapted for use in facing the walls of lock-chambers and in other localities where a stone surface is liable to be rubbed by vessels or other moving bodies. Fig. 36.

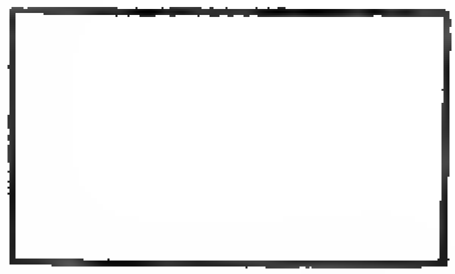

FIG. 36.—RUBBED.

"**Diamond Panels.** Sometimes the space between the margins is sunk immediately adjoining them and then rises gradually until the four planes form an apex at the middle of the panel. In general, such panels are called diamond panels, and the one just described, Fig. 37, is called a sunk diamond panel. When the surface of the stone rises gradually from the inner lines of the margins to the middle of the panel, it is called a raised diamond panel. Both kinds of finish are common on bridge quoins and similar work. The details of this method should be given in the specifications."

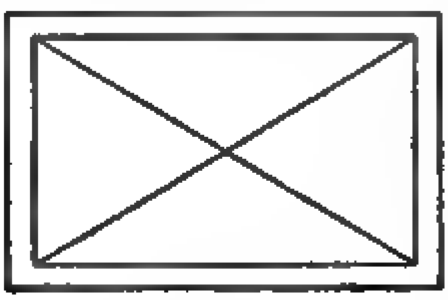

FIG. 37.—DIAMOND PANEL.

CHAPTER VII.

STONE MASONRY.

194. MASONRY is classified (1) according to the nature of the material—as stone masonry, brick masonry, and mixed masonry or that which is composed of stone and brick;—and (2) according to the manner in which the material is prepared—as rubble or unsquared-stone masonry, squared-stone masonry, and ashlar or cut-stone masonry.

The classification of masonry for engineering purposes is based almost entirely upon the size and figure of the stones, and upon the manner in which the joints are formed and executed, the appearance of the face being a matter of secondary importance.

In preparing specifications, it is not safe to depend alone upon the terms in common use to designate the various classes of masonry; but every specification should contain an accurate description of the character and quality of the work desired. Whenever practicable, samples of each kind of cutting and masonry should be prepared beforehand, and be exhibited to the persons who propose to undertake the work.

195. DEFINITIONS OF PARTS OF THE WALL.* *Face,* the front surface of a wall; *back,* the inside surface.

Facing, the stone which forms the face or outside of the wall. *Backing,* the stone which forms the back of the wall. *Filling,* the interior of the wall.

Batter. The slope of the surface of the wall.

Course. A horizontal layer of stone in the wall. If the stones of each layer are of equal thickness throughout, it is termed *regular coursing*; if the thicknesses are unequal, the term *random* or *unequal coursing* is applied.

Joints. The mortar-layer between the stones. The horizontal joints are called *bed-joints* or simply *beds*; the vertical joints are sometimes called the *builds.* Usually the horizontal joints are called *beds,* and the vertical ones *joints.*

* The definitions in this chapter are in accordance with the recommendations of the Committee of the American Society of Civil Engineers previously referred to, and conform to the best practice. Unfortunately they are not universally adopted.

Coping. A projecting course of heavy stones on the top of the wall to protect it.

Pointing. A better quality of mortar put in the face of the joints to help them to resist weathering.

Bond. The arrangement of stones in adjacent courses (§ 202).

Stretcher. A stone whose greatest dimension lies parallel to the face of the wall.

Header. A stone whose greatest dimension lies perpendicular to the face of the wall.

Quoin. A corner-stone. A quoin is a header for one face and a stretcher for the other.

Dowels. Straight bars of iron which enter a hole in the upper side of one stone and also a hole in the lower side of the stone next above.

Cramps. Bars of iron having the ends turned at right angles to the body of the bar, which enter holes in the upper side of adjacent stones.

196. DEFINITIONS OF KINDS OF MASONRY. Ashlar. Cut-stone masonry, or masonry composed of any of the various kinds of cut-stone mentioned in § 193. See Fig. 38. From its derivation, ashlar apparently means large, square blocks; but practice seems to have made it synonymous with "cut-stone," and this secondary meaning has been retained for convenience.

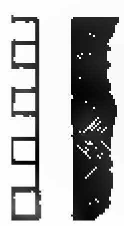

FIG. 38.—ASHLAR.

FIG. 39.—BROKEN ASHLAR.

Broken Ashlar. Cut-stone masonry in which the joints are not continuous. Fig. 39.

Small Ashlar. Cut-stone masonry in which the stones are less than one foot thick. The term is not often used.

Rough Ashlar. A term sometimes given to squared-stone masonry (§ 197), either quarry-faced or pitch-faced, when laid as range-work; but "it is more logical and more expressive to call such masonry squared range-work."

Dimension Stones. Cut-stones, all of whose dimensions have been fixed in advance. "If the specifications for ashlar masonry are so written as to prescribe the dimensions to be used, it will not be necessary to make a new class for masonry composed of such stones."

197. Squared-stone Masonry. Work in which the stones are roughly squared and roughly dressed on beds and joints (§ 192). The distinction between squared-stone masonry and ashlar (§ 196) lies in the degree of closeness of the joints. According to the report of the Committee of the American Society of Civil Engineers, "when the dressing on the joints is such that the distance between the general planes of the surface of adjoining stones is one-half inch or more, the stones properly belong to this class;" nevertheless, such masonry is usually classed as ashlar or cut-stone masonry.

Quarry-faced Masonry. That in which the face of the stone is left as it comes from the quarry. Fig. 40.

Pitched-faced Masonry. That in which the face of the stone is roughly dressed (§ 192, *b*). Fig. 41.

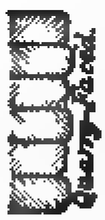

FIG. 40.

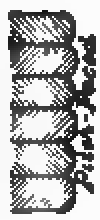

FIG. 41.

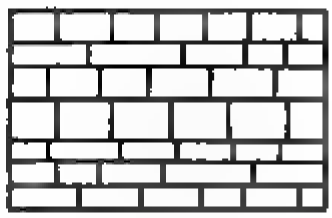

FIG. 42.—RANGE.

Range-work. Masonry in which the course is of the same thickness or rise throughout. Fig. 42.

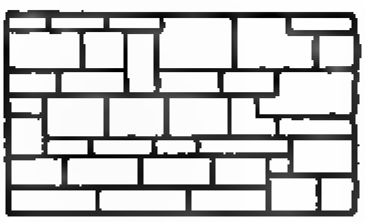

FIG. 43.—BROKEN RANGE.

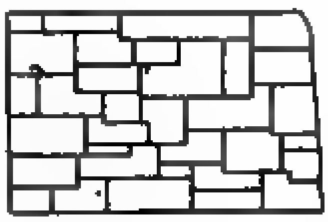

FIG. 44.—RANDOM.

Broken Range-work. Masonry in which the courses are not continuous throughout. Fig. 43.

Random-work. Masonry which is not laid in courses at all. Fig. 44.

198. Rubble Masonry. Masonry composed of unsquared stone (§ 191).

Uncoursed Rubble. Masonry composed of unsquared stones laid without any attempt at regular courses. Fig. 45.

Coursed Rubble. Unsquared-stone masonry which is leveled off at specified heights to an approximately horizontal surface. It may be specified that the stone shall be roughly shaped with the hammer, so as to fit approximately. Fig. 46.

FIG. 45. UNCOURSED RUBBLE.

FIG. 46. COURSED RUBBLE.

199. General Rules. Rankine gives the following rules to be observed in the building of all classes of stone masonry:

"I. Build the masonry, as far as possible, in a series of courses, perpendicular, or as nearly so as possible, to the direction of the pressure which they have to bear; and by breaking joints avoid all long continuous joints parallel to that pressure.

"II. Use the largest stones for the foundation course.

"III. Lay all stones which consist of layers in such a manner that the principal pressure which they have to bear shall act in a direction perpendicular, or as nearly so as possible, to the direction of the layers. This is called *laying the stone on its natural bed,* and is of primary importance for strength and durability.

"IV. Moisten the surface of dry and porous stones before bedding them, in order that the mortar may not be dried too fast and reduced to powder by the stone absorbing its moisture.

"V. Fill every part of every joint, and all spaces between the stones, with mortar, taking care at the same time that such spaces shall be as small as possible."

Another and very important rule is: the rougher the stones, the better the mortar should be. The principal object of the mortar is to equalize the pressure; and the more nearly the stones are reduced to closely fitting surfaces, the less important is the mortar. Not infrequently this rule is exactly reversed; *i. e.*, the finer the dressing, the better the quality of the mortar used.

200. Ashlar Masonry. For definitions of this class of masonry and its subdivision, see § 196.

The strength of a mass of ashlar masonry depends upon the size of the blocks in each course, upon the accuracy of the dressing, and upon the bond.

In order that the stones may not be liable to be broken across, no soft stone, such as the weaker kinds of sandstone and granular limestone, should have a length greater than 3 times its depth; but in harder materials, the length may be 4 or 5 times the depth. The breadth in soft materials may range from 1½ to 2 times the depth; in hard materials, it may be 3 times the depth.

201. Dressing. The closeness with which stones fit is dependent solely upon the accuracy with which the surfaces in contact are wrought, or dressed, and is of special importance in the case of bed-joints. If any part of the surface projects beyond the plane

of the chisel-draft, that projecting part will have to bear an undue share of the pressure, the joint will gape at the edges,—constituting what is called an *open joint*,—and the whole will be wanting in stability. On the other hand, if the surface of the bed is concave, having been dressed down below the plane of the chisel-draft, the pressure is concentrated on the edges of the stone, to the risk of splitting them off. Such joints are said to be *flushed*. They are more difficult of detection, after the masonry has been built, than open joints; and are often executed by design, in order to give a neat appearance to the face of the building. Their occurrence must therefore be guarded against by careful inspection during the progress of the stone cutting.

Great smoothness is not desirable in the joints of ashlar masonry intended for strength and stability; for a moderate degree of roughness adds at once to the resistance to displacement by sliding, and to the adhesion of the mortar. When the stone has been dressed so that all the small ridges and projecting points on its surface are reduced nearly to a plane, the pressure is distributed nearly uniformly, for the mortar serves to transmit the pressure to the small depressions. Each stone should first be fitted into its place dry, in order that any inaccuracy of figure may be discovered and corrected by the stone-cutter before it is finally laid in mortar and settled in its bed.

The thickness of mortar in the joints of the very best ashlar masonry—for example, the United States post-office and custom-house buildings in the principal cities—is about $\frac{1}{8}$ of an inch; in first-class railroad masonry—for example, important bridge piers and abutments, and large arches—the joints are from $\frac{3}{8}$ to $\frac{5}{8}$ of an inch; and in second-class railroad masonry—for example, small bridge piers and abutments, and small arches—the joints do not exceed $\frac{3}{4}$ to 1 inch.

A chisel-draft $1\frac{1}{2}$ or 2 inches wide is usually cut at each exterior corner.

202. Bond. No side-joint of any course should be directly above a side-joint in the course below; but the stones should overlap, or *break joint*, to an extent of from 1 to $1\frac{1}{2}$ times the depth of the course. This is called the *bond* of the masonry. The effect is that each stone is supported by at least two stones of the course below, and assists in supporting at least two stones of the course above. The

object is twofold: first, to distribute the pressure, so that inequalities of load on the upper part of the structure (or of resistance at the foundation) may be transmitted to and spread over an increasing area of bed in proceeding downwards (or upwards); and second, to tie the building together, *i. e.*, to give it a sort of tenacity, both lengthwise and from face to back, by means of the friction of the stones where they overlap.

The strongest bond is that in which each course at the face of the structure contains a header and a stretcher alternately, the outer end of each header resting on the middle of a stretcher of the course below, so that rather more than *one third* of the area of the face consists of ends of headers. This proportion may be deviated from when circumstances require it, but in every case it is advisable that the ends of headers should not form less than *one fourth* of the whole area of the face of the structure. A header should extend entirely through the wall, and should be over the middle of the stretcher in the course below.

A trick of masons is to use "blind-headers," or short stones that look like headers on the outside but do not go deeper into the wall than the adjacent stretchers. When a course has been put on top of these, they are completely covered up; and, if not suspected, the fraud will never be discovered unless the weakness of the wall reveals it.

Where very great resistance to displacement of the masonry is required (as in the upper courses of bridge piers, or over openings, or where new masonry is joined to old, or where there is danger of unequal settlement), the bond is strengthened by dowels or by cramp-irons (§ 195) of, say, 1¼-inch round iron set with cement mortar.

203. Backing. Ashlar is usually backed with rubble masonry (§ 213), which in such cases is specified as coursed rubble. The stones of the ashlar face should have their beds and joints accurately squared and dressed for a distance inward from the face of from 1 to 2 times (say, on the average, 1½ times) the depth of the course; but the backs of these stones may be rough. The proportion and length of the headers should be the same as in ashlar. The "tails" of the headers, or the parts which extend into the rubble backing, may be left rough at the back and sides; but their upper and lower beds should be dressed to the general plane of the bed of

the course. These "tails" may taper slightly in breadth, but should not taper in depth.

The backing should be carried up at the same time with the face-work, and in courses of the same depth; and the bed of each course should be carefully built to the same plane with that of the ashlar facing. The rear face of the backing should be lined to a fair surface.

204. Pointing. In laying masonry of any character, whether with common or hydraulic mortar, the exposed edges of the joints will naturally be deficient in density and hardness. The mortar in the joints near the surface is especially subject to dislodgment, since the contraction and expansion of the masonry is liable either to separate the stone from the masonry or to crack the mortar in the joint, thus permitting the entrance of rain-water, which, freezing, forces the mortar from the joints. Therefore it is usual, after the masonry is laid, to refill the joints as compactly as possible, to the depth of at least half an inch, with mortar prepared especially for this purpose. This operation is called pointing.

The very best cement mortar should be used for pointing, as the best becomes dislodged all too soon. Clear Portland cement mortar is the best, although 1 volume of cement to 1 of sand is frequently used in first-class work. The mortar, when ready for use, should be rather incoherent and quite deficient in plasticity. Before applying the pointing, the joint should be well cleansed by scraping and brushing out the loose matter, and then be well moistened. Of course, the cleansing out of the joints can be most easily done while the mortar is new and soft. The depth to which the mortar shall be dug out is not often specified; it is usually cleaned out about half an inch deep, but should be at least an inch. In the Brooklyn bridge piers the joints were cleared $1\frac{1}{2}$ inches deep.

The mortar is applied with a mason's trowel, and the joint well calked with a calking iron and hammer. In the very best work, the joint is also rubbed smooth with a steel polishing tool. Walls should not be allowed to dry too rapidly after pointing; therefore, pointing in hot weather should be avoided.

205. Amount of Mortar. The amount of mortar required for ashlar masonry varies with the size of the blocks, and also with the closeness of the dressing. With $\frac{3}{8}$- to $\frac{1}{2}$-inch joints and 12- to 20-inch courses, there will be about 2 cubic feet of mortar per

cubic yard; with larger blocks and closer joints, *i. e.*, in the best masonry, there will be about 1 cubic foot of mortar per yard of masonry. See also page 87. Laid in 1 to 2 mortar, ordinary ashlar will require ¼ to ⅓ of a barrel of cement per cubic yard of masonry.

For the quantities of cement and sand required for a cubic yard of mortar of different compositions, see the table on page 86.

206. When Employed. Ashlar masonry is used for piers, abutments, arches, and parapets of bridges; for hydraulic works; for facing-quoins, string courses; for the coping of inferior kinds of masonry and of brickwork; and, in general, for works in which great strength and stability are required.

207. Specifications for Ashlar Masonry. The specifications for ashlar, or "first-class masonry," as employed on railroads,* are about as follows:

First-class masonry shall consist of quarry-faced ashlar, laid in regular horizontal courses, having parallel beds and vertical joints of not less than ten inches (10″), and not more than thirty inches (30″) in thickness, decreasing in thickness regularly from the bottom to the top of the wall.

Stretchers shall not be less than two and one half feet (2½′) nor more than six feet (6′) in length; and not less than one and one half feet (1½′) in width, nor less in width than one and one fourth (1¼) times their depth. Headers must not be less than three and one half feet (3½′) in length, unless the thinness of the wall necessitates it, nor more than four and one half feet (4½′); and not less than one and one half feet (1½′) in width, nor less in width than they are in depth of course. The beds and sides of the stone shall be cut, before being placed on the work, so as to form joints not less than one quarter inch (¼″) and not exceeding five eighths of an inch (⅝″) in width. Every stone must be laid on its natural bed, and all stones must have their beds well dressed, parallel, and true to the proper line, and made always as large as the stone will admit. The vertical joints of the face must not be less than eight inches (8″) in from the face, and as much more as the stone will admit. All corners and batter lines must be built with a neat chisel-draft one and one half inches (1½″) on each corner. The projections of the rock-face must not exceed four inches (4″) beyond the draft-lines of the masonry; in tunnel side walls, this projection must not exceed two inches (2″). The masonry shall consist of headers and stretchers alternating; at least one fourth of the wall shall consist of headers extending entirely through the wall. Every header shall be immediately over a stretcher of the underlying course. The stones of each course shall be so arranged as to form a proper bond with the stones of the underlying course. A bond of less than one foot (1′) will in no case be allowed.

* For complete specifications for railroad and also other kinds of masonry, see Appendix I.

The masonry shall be laid with cement mortar consisting of 1 volume of cement of the Rosendale type and 2 volumes of sand. Each stone shall be cleaned and dampened before being set. No hammering on the wall will be allowed after the course is set; but if any inequalities occur, they must be carefully pointed off.

The backing shall be of good-sized, well-shaped stones, laid so as to break joints and thoroughly bond the work in all directions and leave no spaces between them over six inches (6") wide, which spaces shall be filled with small stones and spalls set in cement mortar.

All foundation courses must be laid with selected, large, flat stones not less than twelve inches (12") in thickness, nor of less superficial surface than fifteen (15) square feet.

The coping shall be formed of large flat stones, which shall extend entirely across the wall when the same is not more than six feet (6') wide. The steps of wing walls shall be capped with stone covering the entire step and extending under the step next above at least twelve inches (12"). Coping and step stones shall be at least twelve inches (12") thick, and have such projections as the engineer may direct [usually 3 to 6 inches]. The tops and faces of copings and step stones shall be bush-hammered, and their joints and beds cut to one quarter inch (¼") throughout.

208. Squared-stone Masonry. For definitions of this class of masonry and its subdivisions, see § 197. The distinction between squared-stone masonry and ashlar lies in the degree of closeness of the joints. According to the Report of the Committee of the American Society of Civil Engineers, "when the dressing on the joints is such that the distance between the general planes of the surfaces of adjoining stones is one half inch or more, the stones properly belong to this class;" however, such masonry is usually classed as ashlar or cut-stone masonry.

Squared-stone masonry is usually quarry-faced, random-work, although range-work is not uncommon. The quoins and the sides of openings are usually *hammer-dressed,* which consists in removing projections so as to secure a rough-smooth surface and is done with the face-hammer, the ordinary ax, or the tooth-ax. This work is a necessity where door or window frames are inserted; and it greatly improves the general effect of the wall, if used wherever a corner is turned.

209. Squared-stone masonry is distinguished, on the one hand, from ashlar in having less accurately dressed beds and joints, and, on the other hand, from rubble in being more carefully constructed. In ordinary practice, the field covered by this class is not very definite. Occasionally the specifications for "second-class masonry"

as used on railroads conform to the above description of pitched-faced, range, squared-stone masonry; but more frequently the term "second-class masonry" is employed to designate "superior rubble." Ordinary rubble is sometimes designated as third-class masonry, but generally simply as rubble. It would be a great improvement upon present practice, if "first-class masonry" were used to designate ashlar or cut-stone masonry, "second-class masonry" for squared-stone masonry, and "third-class masonry" for rubble.

210. Amount of Mortar Required. The amount of mortar required for squared-stone masonry varies with the size of the stones and with the quality of the masonry; as a rough average, one sixth to one quarter of the mass is mortar. For the amount of mortar required for a cubic yard of different classes of masonry, see page 87. When laid in 1 to 2 mortar, squared-stone masonry will require $\frac{1}{2}$ to $\frac{3}{4}$ of a barrel of cement per cubic yard of masonry.

For quantities of cement and sand required for mortars of various compositions, see the table on page 86.

211. Backing and Pointing. See § 203 and § 204, respectively.

212. Specifications for Squared-stone Masonry. Squared-stone masonry is employed for the piers and abutments of the lighter bridges, for arches of 10-foot span and under, for box-culverts, for basement walls, etc. The specifications are usually about as follows:*

The face stones shall have quarry faces with edges pitched to a straight line. Each stone shall be dressed to a uniform thickness, with beds throughout. No stone shall be less than eight inches (8") thick, nor measure in its smallest horizontal dimension less than 12 inches (12"), nor less than its thickness. Joints on the face shall be broken at least eight inches (8"). The bed-joints and also the vertical joints for eight inches (8") back from the face shall be dressed to one inch (1"). The masonry need not be laid up in regular courses, but shall be well bonded, having at least one header, three feet (3') long, to every three stretchers.

The stone shall be laid in full cement mortar beds, and the joints shall be packed full.

The backing shall consist of stone not less than six inches (6") thick. At least one half of the stones in the backing shall measure two (2) cubic feet. The backing shall be laid in full mortar beds; and the vertical joints shall

* For complete specifications for masonry for various purposes, see Appendix I.

also be filled with mortar. The spaces between the large stones shall be filled with spalls set in mortar.

The coping shall be formed of large flat stones of such thickness as the engineer may direct, but in no case to be less than eight inches (8″). The upper surface of the coping shall be bush-hammered, and the joints and beds shall be dressed to one half an inch (½″) throughout. Each stone must extend entirely across the wall when the wall is not more than four feet (4′) thick.

213. Rubble Masonry. For definitions connected with this class of masonry, see § 198.

The stones used for rubble masonry should be prepared by simply knocking off all the weak angles of the block. It should be cleansed from dust, etc., and moistened, before being placed on its bed. This bed is prepared by spreading over the top of the lower course an ample quantity of good, ordinary-tempered mortar in which the stone is firmly embedded. The vertical joints should be carefully filled with mortar. The interstices between the larger masses of stone are filled by thrusting small fragments or chippings of stone into the mortar. In heavy walls of rubble masonry, the precaution should be observed to give the stones the same position in the masonry that they had in the quarry, *i. e.*, to lay them on their "natural bed," since stone offers more resistance to pressure in a direction perpendicular to the quarry-bed than in any other. The directions of the laminæ in stratified stones show the position of the quarry-bed.

To connect the parts well together and to strengthen the weak points, *throughs* or binders should be used in all the courses, and the angles should be constructed of cut or hammered stone.

When carefully executed with good mortar, rubble possesses all the strength and durability required in structures of an ordinary character, and is much less expensive than ashlar. The difficulty is in getting it well executed. The most common defects are (1) not bringing the stones to an even bearing; (2) leaving large vertical openings between the several stones; (3) laying up a considerable height of the wall dry, with only a little mortar on the face and back, and then pouring mortar on the top of the wall; (4) using insufficient cement, or that of a poor quality. The last defect is usually obviated by furnishing the cement to the contractor; and the second and third defects may be detected by probing the vertical joints with a small steel rod. In order to secure good rubble, great skill and

care are required on the part of the mason, and constant watchfulness on the part of the inspector.

A very stable wall can be built of rubble masonry without any dressing, except a draft on the quoins by which to plumb the corners and carry them up neatly, and a few strokes of the hammer to spall off any projections or surplus stone. This style of work is not generally advisable, as very few mechanics can be relied upon to take the proper amount of care in leveling up the beds and filling the joints; and as a consequence, one small stone may jar loose and fall out, resulting probably in the downfall of a considerable part of the wall. Some of the naturally bedded stones are so smooth and uniform as to need no dressing or spalling up; a wall of such stones is very economical, since there is no expense of cutting and no time is lost in hunting for the right stone, and yet strong, massive work is assured. However, many of the naturally bedded stones have inequalities on their surfaces, and in order to keep them level in the course it becomes necessary to raise one corner by placing spalls or chips of stone under the bed, and to fill the vacant spaces well and full with mortar. It is just here that the disadvantage of this style of work becomes apparent. Unless the mason places these spalls so that the stone rests firmly, *i. e.*, does not rock, it will work loose, particularly if the structure is subject to shock, as the walls of cattle-guards, etc. Unless these spalls are also distributed so as to support all parts of the stone, it is liable to be broken by the weight above it. A few such instances in the same work may occasion considerable disaster.

One of the tricks of masons is to put "nigger-heads" (stones from which the natural rounded surface has not been taken off) into the interior of the wall.

214. Rubble masonry is sometimes laid without any mortar, as in slope walls (§ 218), paving (§ 219), etc., in which case it is called dry rubble; but as such work is much more frequently designated as slope-wall masonry and stone-paving, it is better to reserve the term rubble for undressed stone laid in mortar. Occasionally box culverts are built of the so-called dry rubble; but as such construction is not to be commended, there is no need of a term to designate that kind of masonry.

215. Amount of Mortar Required. If rubble masonry is composed of small and irregular stones, about one third of the mass

will consist of mortar; if the stones are larger and more regular, one fifth to one quarter will be mortar. Laid in 1 to 2 mortar, ordinary rubble requires from one half to one barrel of cement per cubic yard of masonry.

For the amount of cement and sand required for mortar of various compositions, see the table on page 86.

216. When Employed. Rubble masonry of the quality described above is frequently employed for the smallest sizes of bridge abutments, small arch culverts, box and open culverts, foundations of buildings, etc., and for backing for ashlar masonry (§ 200).

217. Specifications for Rubble Masonry.* The following requirements, if properly complied with, will secure what is generally known among railroad engineers as superior rubble.

Rubble masonry shall consist of coursed rubble of good quality laid in cement mortar. No stone shall be less than six inches (6") in thickness, unless otherwise directed by the engineer. No stone shall measure less than twelve inches (12") in its least horizontal dimension, or less than its thickness. At least one fourth of the stone in the face shall be headers, evenly distributed throughout the wall. The stones shall be roughly squared on joints, beds, and faces, laid so as to break joints and in full mortar beds. All vertical spaces shall be flushed with good cement mortar and then be packed full with spalls. No spalls will be allowed in the beds. Selected stones shall be used at all angles, and shall be neatly pitched to true lines and laid on hammer-dressed beds; draft lines may be required at the more prominent angles.

The top of parapet walls, piers, and abutments shall be capped with stones extending entirely across the wall, and having a front and end projection of not less than four inches (4"). Coping stones shall be neatly squared, and laid with joints of less than one half inch (½"). The steps of wing-walls shall be capped with stone covering the entire step, and extending at least six inches (6") into the wall. Coping and step stones shall be roughly hammer-dressed on top, their outer faces pitched to true lines, and be of such thickness (not less than six inches) and have such projections as the engineer may direct.

"The specifications for rubble masonry will apply to rubble masonry laid dry, except as to the use of the mortar (see § 214)."

218. Slope-wall Masonry. A slope-wall is a thin layer of masonry used to preserve the slopes of embankments, excavations, canals, river banks, etc., from rain, waves, weather, etc. The usual specifications are as follows:—

The stones must reach entirely through the wall, and be not less than four inches (4") thick and twelve inches (12") long. They must be laid with broken joints; and the joints must be as close and free from spalls as possible.

* For complete specifications for masonry for various purposes, see Appendix I.

219. Stone Paving. Stone paving is used for the inverts of arch culverts, for protecting the lower end of arches from undermining, and for foundations of box culverts and small arches. It is usually classed as dry rubble masonry, although it is occasionally laid with cement mortar. The usual specifications are about as follows:

Stone paving shall be made of flat stones from eight inches (8") to fifteen inches (15") in depth, set on edge, closely laid and well bedded in the soil, and shall present an even top surface.

220. Riprap. Riprap is stone laid, without mortar, about the base of piers, abutments, etc., to prevent scour, and on banks to prevent wash. When used for the protection of piers, the stones are dumped in promiscuously, their size depending upon the material at hand and the velocity of the current; stones of 15 to 25 cubic feet each are frequently employed. When used for the protection of banks, the riprap is laid by hand to a uniform thickness.

221. Strength of Stone Masonry. The results obtained by testing small specimens of stone (see § 13) are useful in determining the relative strength of different kinds of stone, but are of no value in determining the ultimate strength of the same stone when built into a masonry structure. The strength of a mass of masonry depends upon the strength of the stone, the size of the blocks, the accuracy of the dressing, the proportion of headers to stretchers, and the strength of the mortar. A variation in any one of these items may greatly change the strength of the masonry.

The importance of the mortar as affecting the strength of masonry to resist direct compression is generally overlooked. The mortar acts as a cushion (§ 9) between the blocks of stone, and if it has insufficient strength it will be squeezed out laterally, producing a tensile strain in the stone; weak mortar thus causes the stone to fail by tension instead of by compression. No experiments have ever been made upon the strength of stone masonry under the conditions actually occurring in masonry structures, owing to the lack of a testing-machine of sufficient strength. Experiments made upon brick piers (§ 246) 12 inches square and from 2 to 10 feet high, laid in mortar composed of 1 volume Portland cement and 2 sand, show that the strength per square inch of the masonry is only about one sixth of the strength of the brick. An increase of 50 per cent. in the strength of the brick produced no appreciable

effect on the strength of the masonry; but the substitution of cement mortar (1 Portland and 2 sand) for lime mortar (1 lime and 3 sand) increased the strength of the masonry 70 per cent. The method of failure of these piers indicates that the mortar squeezed out of the joints and caused the brick to fail by tension. Since the mortar is the weakest element, the less mortar used the stronger the wall; therefore the thinner the joints and the larger the blocks, the stronger the masonry, provided the surfaces of the stones do not come in contact.

It is generally stated that the working strain on stone masonry should not exceed one twentieth to one tenth of the strength of the stone; but it is clear, from the experiments on the brick piers referred to above, that the strength of the masonry depends upon the strength of the stone only in a remote degree. In a general way it may be said that the results obtained by testing small cubes may vary 50 per cent. from each other (or say 25 per cent. from the mean), owing to undetected differences in the material, cutting, and manner of applying the pressure. Experiments also show that stones crack at about half of their ultimate crushing strength. Hence, when the greatest care possible is exercised in selecting and bedding the stone, the safe working strength of the stone alone should not be regarded as *more* than three eighths of the ultimate strength. A further allowance, depending upon the kind of structure, the quality of mortar, the closeness of the joints, etc., should be made to insure safety. Experiments upon even comparatively large monoliths give but little indication of the strength of masonry. The only practicable way of determining the actual strength of masonry is to note the loads carried by existing structures. However, this method of investigation will give only the load which does not crush the masonry, since probably no structure ever failed owing to the crushing of the masonry. After an extensive correspondence and a thorough search through engineering literature, the following list is given as showing the maximum pressure to which the several classes of masonry have been subjected.

222. Pressure Allowed. Early builders used much more massive masonry, proportional to the load to be carried, than is customary at present. Experience and experiments have shown that such great strength is unnecessary. The load on the monolithic piers supporting the large churches in Europe does not exceed 30

tons per sq. ft. (420 lbs. per sq. in.),* or about one thirtieth of the ultimate strength of the stone alone. The stone-arch bridge of 140 ft. span at Pont-y-Prydd, over the Taff, in Wales, erected in 1750, is supposed to have a pressure of 20.7 tons per sq. ft. (290 lbs. per sq. in.) on hard limestone rubble masonry laid in lime mortar. A former bridge at the same place failed with 64 tons per sq. ft. Rennie subjected good hard limestone rubble in columns 4 feet square to 22 tons per sq. ft. (300 lbs. per sq. in.).† The granite piers of the Saltash Bridge sustain a pressure of 9 tons per sq. ft. (125 lbs. per sq. in.).

The maximum pressure on the granite masonry of the towers of the Brooklyn Bridge is about 28½ tons per sq. ft. (about 400 lbs. per sq. in.). The maximum pressure on the limestone masonry of this bridge is about 10 tons per sq. ft. (125 lbs. per sq. in.). The face stones ranged in cubical contents from 1½ to 5 cubic yards; the stones of the granite backing averaged about 1½ cu. yds., and of the limestone about 1¼ cu. yds. per piece. The mortar was 1 volume of Rosendale cement and 2 of sand. The stones were rough-axed, or pointed to ¼-inch bed-joints and ½-inch vertical face-joints.‡ These towers are very fine examples of the mason's art.

In the Rookery Building, Chicago, granite columns about 3 feet square sustain 30 tons per sq. ft. without any signs of weakness.

In the Washington Monument, Washington, D. C., the normal pressure on the lower joint of the walls of the shaft is 20.2 tons per sq. ft. (280 lbs. per sq. in.), and the maximum pressure brought upon any joint under the action of the wind is 25.4 tons per sq. ft. (350 lbs. per sq. in.).§

The pressure on the limestone piers of the St. Louis Bridge was, before completion, 38 tons per sq. ft. (527 lbs. per sq. in.); and after completion the pressure was 19 tons per sq. ft. (273 lbs. per sq. in.) on the piers and 15 tons per sq. ft. (198 lbs. per sq. in.) on the abutments.‖

The limestone masonry in the towers of the Niagara Suspension

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* In this connection it is convenient to remember that 1 ton per square foot is equivalent nearly to 14 (exactly 13.88) pounds per square inch.

† Proc. Inst. of C. E., vol. x. p. 241.

‡ F. Collingwood, asst. engineer, in Trans. Am. Soc. of C. E.

§ Report of Col. T. L. Casey, U. S. A., engineer in charge.

‖ History of St. Louis Bridge, pp. 370–74.

Bridge failed under 36 tons per sq. ft., and were taken down,—however, the masonry was not well executed.*

At the South Street Bridge, Philadelphia, the pressure on the limestone rubble masonry in the pneumatic piles is 15.7 tons per sq. ft. (220 lbs. per sq. in.) at the bottom and 12 tons per sq. ft. at the top. "This is unusually heavy, but there are no signs of weakness."† The maximum pressure on the rubble masonry (laid in cement mortar) of some of the large masonry dams is from 11 to 14 tons per sq. ft. (154 to 195 lbs. per sq. in.). The Quaker Bridge Dam is designed for a maximum pressure of 16⅔ tons per sq. ft. (230 lbs. per sq. in.) on massive rubble masonry in best hydraulic cement mortar.‡

223. Safe Pressure. In the light of the preceding examples it may be assumed that the safe load for the different classes of masonry is about as follows, provided each is the best of its class:

Concrete, ...	5 to 15 tons per square foot.
Rubble, ...	10 to 15 " " " "
Squared stone, ...	15 to 20 " " " "
Limestone ashlar, ...	20 to 25 " " " "
Granite ashlar, ...	30 " " " "

224. Measurement of Masonry. The method of determining the quantity of masonry in a structure is frequently governed by trade rules or local custom, and these vary greatly with locality. Masons have voluminous and arbitrary rules for the measurement of masonry; for example, the masons and stone-cutters of Boston at one time adopted a code of thirty-six complicated rules for the measurement of hammer-dressed granite. As an example of the indefiniteness and arbitrariness of all such rules, we quote the following, which are said to be customary in Pennsylvania: "All openings less than 3 feet wide are counted solid. All openings more than 3 feet wide are taken out, but 18 inches is added to the running measurement for every jamb built. Arches are counted solid from the spring of the arch, and nothing allowed for arching. The corners of buildings are measured twice. Pillars less than 3 feet square are counted on three sides as lineal measurement, multiplied by the fourth side and depth; if more than 3 feet, the two opposite

* Trans. Am. Soc. of C. E., vol. xvii. pp. 204–12. † *Ibid.*, vol. vii. pp. 305–6.
‡ *Engineering News*, vol. xix. p. 75.

sides are taken; to each side 18 inches for each jamb is added to lineal measurement thereof; the whole multiplied by the smaller side and multiplied by the depth."

A well-established custom has all the force of law, unless due notice is given to the contrary. The more definite, and therefore better, method is to measure the exact solid contents of the masonry, and pay accordingly. In "net measurement" all openings are deducted; in "gross measurement" no openings are deducted.

The quantity of masonry is usually expressed in cubic yards. The perch is occasionally employed for this purpose; but since the supposed contents of a perch vary from 16 to 25 cubic feet, the term is very properly falling into disuse. The contents of a masonry structure are obtained by measuring to the neat lines of the design. If a wall is built thicker than specified, no allowance is made for the masonry outside of the limiting lines of the design; but if the masonry does not extend to the neat lines, a deduction is made for the amount it falls short. Of course a reasonable working allowance must be made when determining whether the dimensions of the masonry meet the specifications or not.

In engineering construction it is a nearly uniform custom to measure all masonry in cubic yards; but in architectural construction it is customary to measure water tables, string-courses, etc., by the lineal foot, and window-sills, lintels, etc., by the square foot. In engineering, all dressed or cut-stone work, such as copings, bridge seats, cornices, water-tables, etc., is paid for in cubic yards, with an additional price per square foot for the surfaces that are dressed, cut, or bush-hammered.

225. Classification of Railroad Masonry. The stone masonry required in the construction of a railroad is usually classified about as follows: first-class masonry, second-class masonry, rubble masonry (sometimes called third-class masonry, § 209), rubble masonry laid dry (§ 214), stone paving, slope-walls, and riprap. First-class masonry is equivalent to ashlar (§§ 200–7); this head generally includes bridge abutments and piers of the larger class, and arch culverts of greater span than 10 feet. Sometimes second-class masonry is specified as squared-stone masonry (§§ 208–12), and sometimes as superior rubble (§§ 213–17); it is used in less important structures than first-class masonry.

Frequently specifications recognize also the following classifica-

tion: first-class arch masonry, second-class arch masonry, first-class bridge-pier masonry, second-class bridge-pier masonry, and pedestal masonry. The quality of work thus specified is the same as for first-class and second-class masonry respectively, the only difference being peculiar to the form of the masonry structure, as will be discussed in succeeding chapters. The specifications for each structure should give the quantities of each kind of masonry.

For complete specifications for railroad masonry, see Appendix I.

226. Estimates of Cost of Masonry. The following estimates of the cost of masonry, from Trautwine's *Engineer's Pocket-book*,* are pronounced by experts to be as accurate as such averages can be stated, since every item is liable to great variation. The estimates are based on the assumption that a mason receives $3.50 and a laborer $2.00 per day of 8 hours.

227. "**Quarrying.** After the preliminary expenses of purchasing the site of a good quarry, cleaning off the surface earth and disintegrated top rock, and providing the necessary tools, trucks, cranes, etc., the total net expenses for *getting out* the rough stone for masonry ready for delivery may be roughly estimated thus: Stones of such size as two men can readily lift, measured in piles, will cost per cubic yard from ¼ to ⅓ the daily wages of a quarry laborer. Large stones, ranging from ⅓ to 1 cubic yard each, got out by blasting, from 1 to 2 daily wages per cubic yard. Larger stones, ranging from 1 to 1⅓ cubic yards each, in which most of the work must be done by wedges in order that the individual stones shall come out in tolerably regular shape and conform to stipulated dimensions, from 2 to 4 daily wages per cubic yard. The lower prices are low for sandstone, while the higher ones are high for granite. Under ordinary circumstances, about 1⅓ cubic yards of good sandstone can be quarried at the same cost as 1 of granite—or, in other words, calling the cost of granite 1, that of sandstone will be ¾; hence the means of the foregoing limits may be regarded as rather full prices for sandstone, rather scant for granite, and about fair for limestone or marble.

228. "**Dressing.** In the first place, a liberal allowance should be made for waste. Even when the stone wedges out handsomely on all sides in large blocks of nearly the required shape and size,

* Published by permission.

from $\frac{1}{6}$ to $\frac{1}{4}$ of the rough block will generally not more than cover waste of dressing. In moderate-sized blocks (say averaging about $\frac{1}{2}$ a cubic yard each) got out by blasting, from $\frac{1}{4}$ to $\frac{1}{3}$ will not be too much for stone of medium character as to straight splitting. The last allowance is about right for well-scabbled dressing. The smaller the stones the greater must be the allowance for waste. In large operations it becomes expedient to have the stones dressed, as far as possible, at the quarry, in order to diminish the cost of transportation, which, when the distance is great, constitutes an important item—especially when by land and on common roads.

229. "**Ashlar.** Average size of the stones, say 5 feet long, 2 feet wide, and 1.4 feet thick—or two such stones to a cubic yard. Then, supposing the stone to be of granite or gneiss, the cost per cubic yard of ashlar facing will be:

"Getting out the stone from the quarry by blasting, allowing $\frac{1}{4}$ for waste in dressing, $1\frac{1}{3}$ cubic yards at $3.00 per yard,	$4 00
Dressing 14 sq. ft. of face at 35 cents,	4 90
Dressing 52 sq. ft. of beds and joints at 18 cents,	9 36
Net cost of the dressed stone at the quarry,	$18 26
Hauling (say 1 mile), loading, and unloading,	1 20
Mortar, say,	40
Laying, including scaffold, hoisting machinery, etc.,	2 00
Net cost,	$21 86
Profit to contractor, say 15 per cent.,	3 28
Total cost per cubic yard,	$25 14

"Dressing will cost more if the faces are to be rounded or moulded. If the stones are smaller than we have assumed, there will be more square feet per cubic yard to be dressed. If, in the foregoing case, the stones be perfectly well dressed on all sides, including the back, the cost per cubic yard would be increased about $10; and if some of the sides be curved, as in arch stones, say $12 or $14; and if the blocks be carefully wedged out to given dimensions, $16 or $18. Under these conditions the net cost of the dressed stone *at the quarry* will be $28, $31, and $35 per cubic yard, respectively.

"If the stone be sandstone with good natural beds, the getting out may be put at $3.00 per cubic yard. Face dressing at 26 cents

per sq. ft., say $3.64 per cu. yd. Beds and joints at 13 cents per sq. ft., say $6.76 per cu. yd. The total cost, then, is $19.55 instead of $25.14 for granite, and the net cost $17.00 instead of the $21.86 per cu. yd. for granite. The total cost of large, well-scabbled, ranged sandstone masonry in mortar may be taken at about $10 per cu. yd.

230. "Rubble. With stones averaging about ¼ cubic yard each, and common labor at $1 per day, the cost of *granite rubble*, such as is generally used as backing for the foregoing ashlar, will be about as follows:

Getting out the stone from the quarry by blasting, allowing ¼ for waste in scabbling, 1¼ cu. yds. @ $3.00,	$3 48
Hauling 1 mile, loading and unloading,	1 20
Mortar (2 cu. ft., or 1.6 struck bushels of quicklime, and 10 cu. ft. or 8 struck bushels of sand or gravel, and mixing),	1 50
Scabbling, laying, scaffolding, hoisting machinery, etc.,	2 50
Net cost,	$8 68
Profit to contractor, say 15 per cent.,	1 30
Total cost per cubic yard,	$9 98

"*Common rubble* of small stones, the average size being such as two men can handle, costs to get it out of the quarry about 80 cts. per yard of pile, or, to allow for waste, say $1.00. Hauling 1 mile, $1.00. It can be roughly scabbled and laid for $1.20 more. Mortar, as above, $1.50. Total net cost, $4.70; or with 15 per cent. profit, $5.40, at the above wages for labor."

231. Market Price of Stone. The average market quotations to builders and contractors for the year 1888 were about as follows, *f.o.b.* (free on board) at the quarry:

Granite—rough,	$0 40 to	$0 50	per cubic foot.
Limestone—common rubble,	1 00 "	1 50	per cubic yard.
" good range rubble,	1 50 "	2 00	" " "
" bridge stone,	08 "	10	per cubic foot.
" dimension stone,	25 "	35	" " "
" copings,	20 "	35	" " "
Sandstone,	85 "	1 00	per cubic yard.

232. Actual Cost. In U. S. Public Buildings. The following table gives the average contract price during the past few years for cutting the stone for the United States government buildings: *

* American Architect, vol. xxii. pp. 6, 7.

TABLE 15.

COST OF CUTTING STONE FOR U. S. PUBLIC BUILDINGS.

KIND OF SURFACE.	GRANITE.		MARBLE.		LIMESTONE AND SANDSTONE.	
	Min.	Max.	Min.	Max.	Min.	Max.
Beds and joints, per sq. ft....	$0 30	$0 35	$0 20	$0 25	$0 12	$0 15
Pean-hammered, " " " ...	45	50	30	35	15	20
Plain face, 6-cut, " " " ...	....	65	....	....	....	....
" " 8-cut, " " " ...	....	75	....	....	....	....
" " 10-cut, " " " ...	....	88	....	....	....	....
" " 12-cut, " " " ...	....	1 10	....	....	....	....
Rubbed, " " " ...	....	....	....	40	20	25
Tooled, " " " ...	....	....	....	50	25	30

The following table shows the contract price for the masonry of the United States public buildings :

TABLE 16.

COST OF MASONRY IN U. S. PUBLIC BUILDINGS.

KIND OF WORK.	PLACE.	DATE.	COST PER CU. FT.
Random rubble, limestone................	Harrisburg, Va....	1885	$0 20
" " "	Cincinnati, O.....	1884	20
" " "	Denver, Col.......	1883	20
" " sandstone................	Pittsburgh, Pa....	1886	35
Squared masonry, sandstone..............	" "	1885	60
Coursed masonry, sandstone..............	" "	1885	70
Squared masonry, limestone..............	Columbus, O......	1884	68
" " granite............. ...	Memphis, Tenn....	1886	80
Rock-face ashlar, "	Pittsburgh, Pa....	1886	1 38
" " " and cut-stone granite, avg.	" "	1886	1 60
Cut granite, basement and area walls......	" "	1886	2 00
Rock-face ashlar, and cut and moulded trimmings, Stony Point, Mich., sandstone..	Fort Wayne, Ind..	1885	1 52
Trimmings. Bedford limestone. bid........	" " " ..	1885	1 65
Rock-face ashlar, granite, retaining wall...	Memphis, Tenn...	1886	1 00
Dressed coping, " " " ...	" " ...	1886	2 50
White sandstone,—furnished only...	Dallas, Tex........	1885	35
Armijo " " "	Denver, Col.......	1885	73
Cut and moulded sandstone of superstructure	Council Bluffs, Ia.	1885	1 91
" " " " average bid....	" " "	1885	2 12
" " " limestone, lowest bid.....	" " "	1885	1 87
" " " " average bid.....	" " "	1885	2 33
Rock-face ashlar, cut and moulded trimmings, Middlesex brownstone.........	Rochester, N. Y...	1884	2 41
Cut and moulded, Bedford limestone......	Louisville, Ky.....	1885	2 00
" " ", sandstone.............	Dallas, Tex........	1885	2 46
" " " limestone.............	Hannibal, Mo......	1885	1 88
" " " sandstone.............	Des Moines, Ia....	1887	2 27
" " " granite, superstructure..	Pittsburgh, Pa.....	1886	3 00

233. Railroad Masonry. The following are the average prices actually paid in the construction of the Cincinnati Southern Railroad, in 1873–77:*

First-class bridge masonry, per cu. yd.,	$10 39
Second-class bridge masonry, *in cement*, per cu. yd.,	7 40
Second-class bridge masonry, *dry*, per cu. yd.,	7 02
First-class arch masonry, per cu. yd.,	11 24
Second-class arch masonry, *in cement*, per cu. yd.,	8 61
Second-class arch masonry, *dry*, per cu. yd.,	7 75
Brick-work in tunnels, per cu. yd.,	8 50
Brick-work in buildings, per cu. yd.,	7 00
Box-culvert masonry, *in cement*, per cu. yd.,	4 89
Box-culvert masonry, *dry*, per cu. yd.,	4 32
Concrete, per cu. yd.,	5 52
Slope walls, per cu. yd.,	4 41
Stone paving, per cu. yd.,	2 41

234. Tunnel Masonry. The following are the average prices† paid in 1883–87 on the new Croton Aqueduct tunnel which supplies New York City with water. The mortar was 2 sand to 1 Rosendale cement.

Dimension-stone masonry (granite),	$42 50
Brick-work lining, per cu. yd.,	10 14
Brick-work backing, per cu. yd.,	8 49
Rubble masonry, lining, per cu. yd.,	5 05
Concrete lining, 3 stone to 1 Rosendale cement, per cu. yd.,	5 67
Concrete lining, 5 stone to 1 Rosendale, per cu. yd.,	5 16
Concrete backing, 3 stone to 1 Rosendale, per cu. yd.,	4 73
Concrete backing, 5 stone to 1 Rosendale, per cu. yd.,	4 22
Fine-hammered face (6-cut) for cut stone, per sq. ft.,	84
Rough-pointed face for cut stone, per sq. ft.,	50
Additional for all kinds of masonry laid in Portland cement mortar, 2 to 1, per cu. yd.,	1 78
Additional for all kinds of masonry laid in Rosendale cement mortar, 1 to 1, per cu. yd.,	1 20

235. Bridge-pier Masonry. The following are the details of the cost, to the contractor, of heavy first-class limestone masonry for bridge-piers erected in 1887 by a prominent contracting firm:

* Report of the Chief Engineer, December 1, 1877, Exhibit 3.
† Report of the Commissioners, Table 4.

Cost of stone (purchased),	$4 50
Sand and cement,	52
Freight,	1 79
Laying,	1 40
Handling materials,	65
Derricks, tools, etc.,	40
Superintendence, office expense, etc.,	68
Total cost per cubic yard,	$9 94

The following data concerning the cost in 1887 of granite piers—two fifths cut-stone facing and three fifths rubble backing—are furnished by the same firm. The rock was very hard and tough.

Facing:—

Quarrying, including opening quarry,	$3 75
Cutting to dimensions,	6 75
Laying,	1 76
Transportation 2 miles, superintendence, and general expenses,	2 05
Total cost per cubic yard,	$14 31

Backing:—

Quarrying,	$3 10
Dressing,	3 60
Laying,	1 75
Sundries,	2 05
Total cost per cubic yard,	$10 50

The first-class limestone masonry in the piers of the bridges across the Missouri at Plattsmouth (1879–80) cost the company $18.60 per cubic yard, exclusive of freight, engineering expenses, and tools.* The cost of first-class masonry in smaller piers usually ranges from $12 to $14 per cubic yard.

At Chicago in 1887 the contract price for the masonry in bridge piers and abutments was about as follows: Concrete, 1 Portland cement, 3 sand, 6 broken stone, $9.00 per cu. yd.; concrete, 1 Rosendale cement, 3 sand, 5 broken stone, $6.00 per cu. yd.; stone facing and coping, $30.00 per cu. yd.

236. Arch-culvert Masonry. The following are the details of the cost of the sandstone arch culvert (613 cu. yds.) at Nichols Hollow, on the Indianapolis, Decatur and Springfield Railroad,

* Report of the Chief Engineer, Geo. S. Morison.

built in 1887. Scale of wages per day of 10 hours—foreman, \$3.50; cutters, \$3.00; mortar mixer, \$1.50; laborer, \$1.25; water-boy, 50 cents; carpenters, \$2.50. †

TABLE 17.

ACTUAL COST OF ARCH MASONRY ON INDIANAPOLIS, DECATUR AND SPRINGFIELD RAILROAD.

Items.	Cost. Total.	Cost. Per cu. yd.
Materials:—		
Stone—613 cu. yds. of sandstone @ \$1 50	\$919 50	\$1 50
Cement—180 bbls. German Portland @ \$3 17 = \$412 50		
40 " English " @ 3 25 = 130 00		
30 " Louisville " @ 96 = 28 75	571 25	94
Sand—7 car-loads @ \$5 50	38 50	06
Total for materials	\$1.529 25	\$2 50
Cutting:—		
Cutters and helpers	\$1,370 48	\$2 24
Templates, bevels, straight-edges, etc	11 00	01
Repairs of cutters' tools	52 39	09
Water-boy	11 75	02
Total for cutting	\$1,445 62	\$2 36
Laying:—		
Masons, 110 days @ \$3.50	\$384 87	\$0 63
Masons' helpers	458 66	74
Mortar mixer	121 72	20
Water-boy	11 75	02
Arch centers, building and erecting	37 65	06
Derrick, stone chute, etc	14 63	02
Laying track	7 70	01
Total for laying	\$1.032 08	\$1 68
Pointing	\$30 00	\$0 05
GRAND TOTAL:		
Total for labor	\$2,507 60	\$4 09
Total for materials	1,529 25	2 50
Total cost of masonry	\$4,036 85	\$6 59

238. Summary of Cost. The following table, compiled from a large amount of data, will be convenient for hasty reference. Of course any such table must be used with caution, since such items are subject to great variation.

† Data furnished by Edwin A. Hill, chief engineer.

TABLE 18.

SUMMARY OF COST OF MASONRY.

DESCRIPTION OF MASONRY.	COST PER CUBIC YARD.		
	Min.	Max.	Average.
Arch masonry, first-class	$7 00	$12 00	$10 00
Arch masonry, second-class (in cement)	5 00	10 00	8 00
Box-culvert masonry, in cement	2 50	5 00	3 50
Brick masonry (see § 258)	6 00	10 00	8 00
Bridge masonry, first-class	10 00	20 00	14 00
Bridge masonry, second-class (in cement)	6 00	12 00	10 00
Concrete	2 50	6 00	4 00
Coping	8 00	14 00	12 00
Dimension-stone masonry, granite	40 00	60 00	50 00
Paving	1 00	4 00	2 00
Slope-wall masonry	2 00	5 00	3 00
Squared-stone masonry	6 00	10 00	7 00
Riprap	1 00	2 50	1 50
Rubble, first-class	4 00	6 00	5 00
Rubble, second-class (in cement)	2 00	5 00	3 00

CHAPTER VIII.

BRICK MASONRY.

239. Mortar. Lime mortar is generally employed for brick masonry, particularly in architectural constructions. Many of the leading railroads lay all brick masonry in cement mortar, and the practice should be followed more generally. The weakest part of a brick structure is the mortar. The primary purpose of the mortar is to form an adhesive substance between the bricks; the second is to form a cushion to distribute the pressure uniformly over the surface. If the mortar is weaker than the brick, the ability of the masonry to resist direct compression is thereby considerably reduced. For the reason, see § 9; for the amount, see the Table 19, page 164.

If the strains upon a wall were only those arising from a direct pressure, the strength of the mortar would in most cases be of comparatively little importance, for the crushing strength of average quality mortar is far higher than the dead load which under ordinary circumstances is put upon a wall; but, as a matter of fact, in buildings the load is rarely that of a direct crushing weight, other and more important strains being developed by the system of construction. Thus the roof tends to throw the walls out, the rafters being generally so arranged as to produce a considerable outward thrust against the wall. The action of the wind also produces a side strain which is practically of more importance than either of the others. In many cases the contents of a building exert an outward thrust upon the walls; for example, barrels piled against the sides of a warehouse produce an outward pressure against the walls.

In many brick constructions the use of cement mortar is absolutely necessary—as, for example, in tall chimneys, where the bearing is so small that great strength of the cementing material is required.

240. The thickness of the mortar-joints should be about $\frac{1}{4}$ to $\frac{3}{8}$ of an inch. Thicker joints are very common, but should be avoided. If the bricks are even fairly good, the mortar is the weaker part of

the wall; hence the less mortar the better. Besides, a thin layer of mortar is stronger under compression than a thick one (see § 14). The joints should be as thin as is consistent with their insuring a uniform bearing and allowing rapid work in spreading the mortar. The joints of outside walls should be thin in order to decrease the disintegration by weathering. The joints of inside walls are usually made from ⅜ to ½ inch thick.

Brick should not be merely *laid*, but every one should be rubbed and pressed down in such a manner as to force the mortar into the pores of the bricks and produce the maximum adhesion; with quick-setting cement this is still more important than with lime mortar. For the best work it is specified that the brick shall be laid with a "shove joint;" that is, that the brick shall first be laid so as to project over the one below, and be pressed into the mortar, and then be shoved into its final position.

Lime mortar is liable to work out of the joints, owing to the action of the elements and to changes of temperature. Hence it is customary either (1) to lay the face in mortar containing more lime than that used for the interior, or (2) to lay the face in a mortar containing more or less cement, or (3), in rare cases, to point the joints with neat cement mortar. Whatever the kind of mortar used, the finish of the face of the joint is important. The most durable joint is finished as shown in Fig. 47, although, unfortunately for durability, it is customary to make the slope in the opposite direction.

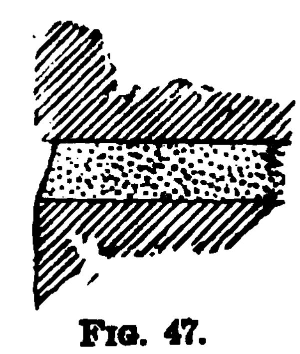


FIG. 47.

241. Since brick have great avidity for water, it is best to dampen them before laying. If the mortar is stiff and the brick dry, the latter absorb the water so rapidly that the mortar does not set properly, and will crumble in the fingers when dry. Neglect in this particular is the cause of most of the failures of brick-work. Since an excess of water in the brick can do no harm, it is best to thoroughly drench them with water before laying. Lime mortar is sometimes made very thin, so that the brick will not absorb all the water. This process interferes with the setting of the mortar, and particularly with the adhesion of the mortar to the brick. Watery mortar also contracts excessively in drying (if it ever does dry), which causes undue settlement and, possibly, cracks or distortion. Wetting the brick before laying will also remove the dust from the surface, which otherwise would prevent perfect adhesion.

242. Bond. The bricks used in a given wall being of uniform size are laid according to a uniform system, which is called the *bond* of the brick-work. As in ashlar masonry, so in brick-work, a *header* is a brick whose length lies perpendicular to the face of the wall; and a *stretcher* is one whose length lies parallel with the face. Brick should be made of such a size that two headers and a mortar-joint will occupy the same length as a stretcher.

243. English Bond. This consists in laying entire courses of headers and stretchers, which sometimes alternate, as in Fig. 48; but generally only one course of headers is laid for every two, three, four, etc., courses of stretchers. In ordinary practice the custom is to lay four to six courses of stretchers to one of headers. The stretchers bind the walls together lengthwise; the headers, crosswise. The proportionate numbers of the courses of headers and stretchers should depend on the relative importance of transverse and longitudinal strength. The proportion of one course of headers to two of stretchers is that which gives equal tenacity to the wall lengthwise and crosswise.

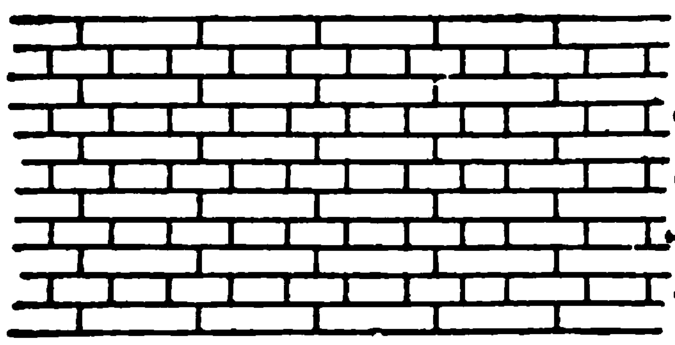


Fig. 48.—English Bond.

In building brick-work in English bond, it is to be borne in mind that there are twice as many vertical or side joints in a course of headers as there are in a course of stretchers; and that unless in laying the headers great care be taken to make these joints very thin, two headers will occupy a little more space than one stretcher, and the correct breaking of the joints—exactly a quarter of a brick—will be lost. This is often the case in carelessly built brick-work, in which at intervals vertical joints are seen nearly or exactly above each other in successive courses.

244. Flemish Bond. This consists of a header and a stretcher alternately in each course, so placed that the outer end of each header lies on the middle of a stretcher in the course below (Fig. 49). The number of vertical joints in each course is the same, so that there is no risk of the correct breaking of the joints by a quarter of a brick being lost; and the wall presents a neater appearance than one built in

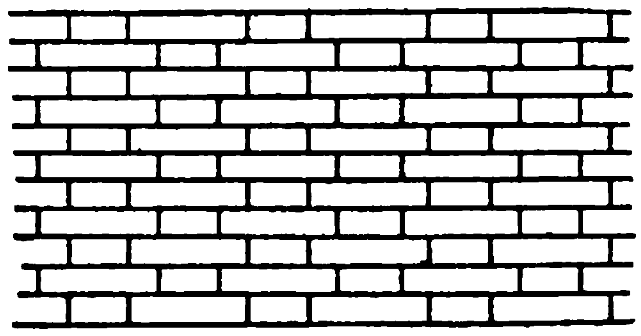


Fig. 49.—Flemish Bond.

English bond. The latter, however, when correctly built, is stronger and more stable than Flemish bond.

245. Hoop-iron Bond. Pieces of hoop-iron are frequently laid flat in the bed-joints of brick-work to increase its longitudinal tenacity, about 2 inches of the ends of each piece being bent down and inserted into the vertical joints. Although thin strips of iron are generally employed, it would be better to use thicker pieces; the value of the iron for this purpose depends wholly upon the rigidity of the ends which are turned down, and this will vary about as the square of the thickness. The strip of iron should be nearly as thick as the mortar-joint. This means of strengthening masonry is frequently employed over openings and to connect interior brick walls with stone fronts.

246. Compressive Strength of Brick Masonry. Experiments at Watertown, Mass., with the United States testing-machine, upon piers 12 inches square and from 1 ft. 4 in. to 10 ft. high, gave results as follows:*

TABLE 19.

STRENGTH OF BRICK MASONRY COMPARED WITH THAT OF THE BRICK AND THE MORTAR.

Reference No.	Composition of the Mortar.	Number of Experiments.	Ultimate Strength of the Pier, in lbs. per sq. inch.	Strength of the Mortar (6-inch Cubes crushed between Steel) in lbs. per sq. in. Mean of three Trials.	Strength of the Pier in terms of the Strength of the Brick.			Strength of the Pier in Terms of the Strength of the Mortar.
					Min.	Max.	Mean.	
1	1 lime, 3 sand	15	1,508	124	.06	.18	.10	12
2	2 mortar (1 lime, 3 sand), 1 Rosendale cement	1	1,646	183	....	....	.11	9
3	2 mortar (1 lime, 3 sand), 1 Portland cement	1	1,411	192	....	....	.09	7
4	1 Rosendale cement, 2 sand	1	1,972	162	....	....	.13	12
5	1 Portland cement, 2 sand	8	2,544	545	.10	.27	.17	4.7
6	Clear Rosendale	....	.....	521	....	....	....	....
7	Clear Portland cement	1	2,375	3,483	....	....	.16	0.7

* Report on "Tests of Metals, etc.," for the year ending June 30, 1884, pp. 69–122.

The brick had an average strength of nearly 15,000 lbs. per sq. in., tested flatwise between steel. The mortar was 14½ months old when it was tested. The piers were built by a common mason, with only ordinary care; and they were from a year and a half to two years old when tested. Their strength varied with their height; and in a general way the experiments show that the strength of a prism 10 ft. high, laid in either lime or cement mortar, is about two thirds that of a 1-foot cube. A deduction derived from so few experiments (22 in all) is not, however, conclusive. The different lengths of the piers tested occurred in about equal numbers. The piers began to show cracks at one half to two thirds of their ultimate strength.

In attempting to draw conclusions from any experiments, it must be borne in mind continually that the result of a single trial may possibly be greatly in error. In this case this precaution is very important, since the difference between experiments apparently exactly alike was in some cases as much as 50 per cent. A great variation in the results is characteristic of all experiments on stone, brick, mortar, etc. Except on the ground of a variation in experiments, it is difficult to explain why mortar No. 4 is weaker than No. 2, while the masonry is stronger; or why the masonry of No. 5 is stronger than that of No. 7.

Of course the apparent efficiency of the masonry, as given in the table, depends upon the manner in which the strengths of the brick and mortar were determined, as well as upon the method of testing the masonry. For example, if the brick had been tested on end the apparent efficiency of the masonry would have been considerably more; or if the mortar had been tested in thin sheets the strength of the masonry relative to that of the mortar would not have been so great.*

247. Some German experiments† gave results as in the table

* It should be mentioned that the mortar with which these piers were built appears to be much weaker than similar mortar under like conditions. (Compare page 72, and pages 126, 166, 188, 197 of the Report of Tests of Metals, etc., made at Watertown in 1884.) Ordinarily, mortar is eight to ten times as strong in compression as in tension, whereas the first six mortars in the preceding table were but little stronger in compression than such mortar should have been in tension. The officer in charge is "unable to offer any explanation. The cement was bought on the market; the maker's name is not known. The cement was not tested." However, the experiments are consistent with themselves, and therefore show relative strengths correctly.

† Van Nostrand's Engin'g Mag., vol. xxxiv. p. 240, from the Abstracts of the Inst. of C. E. (London).

below. It is not stated how the strength of the brick or of the masonry was determined. The term cement in the table refers to Portland cement. According to the building regulations of Berlin, the safe load for brick masonry is less than one tenth of the results in the table.

TABLE 20.

RELATIVE STRENGTH OF BRICK AND BRICK MASONRY.

KIND OF BRICK.	AVERAGE CRUSHING STRENGTH OF BRICK, IN LBS. PER SQ. IN.	ULTIMATE STRENGTH, IN LBS. PER SQ. IN., OF BRICK-WORK WITH MORTAR COMPOSED OF—			
		1 Lime, 2 Sand.	7 Lime, 1 Cement, 16 Sand.	1 Cement, 6 Sand.	1 Cement, 3 Sand.
Ordinary stock........	2,930	1,290	1,890	1,610	1,850
Selected "	3,669	1,620	1,760	2,020	2,320
Clinker "	5,390	2,370	2,590	2,960	3,410
Porous...............	2,617	1,150	1,250	1,440	1,650
Porous perforated.....	1,195	530	570	650	750
Perforated............	2,759	1,210	1,320	1,520	1,710
Average strength of the masonry in terms of the strength of the brick...		0.44	0.48	0.55	0.63

Both of the preceding series of experiments show conclusively that the strength of brick masonry is mainly dependent upon the strength of the mortar. An increase of 50 per cent. in the strength of the brick shows no appreciable effect on the strength of the masonry. Notice, however, that the masonry in the fifth line of Table 19 is 70 per cent. stronger than that in the first, due to the difference between a good Portland cement mortar and the ordinary lime mortar. In the second table notice that brick laid in a 1 to 3 Portland cement mortar is nearly 50 per cent. stronger than in a 1 to 2 lime mortar. Similar experiments* show that masonry laid in mortar composed of 1 part Rosendale cement and 2 parts sand is 56 per cent. stronger than when laid in mortar composed of 1 part lime and 4 parts sand. A member of the Institute of Civil Engineers (London) says† that brick-work laid in lime is only one fourth as strong as when laid in clear Portland cement. Probably the difference in durability between cement mortar and lime mortar is considerably greater than their difference in strength.

* Report of Experiments on Building Materials for the City of Philadelphia with the U. S. testing-machine at Watertown, Mass., pp. 32, 33.

† Proc. Inst. of C. E., vol. xvii. p. 441.

248. Pressure allowed in Practice. The pressure at the base of a brick shot-tower in Baltimore, 246 feet high, is estimated at $6\frac{1}{2}$ tons per sq. ft. (about 90 lbs. per sq. in.). The pressure at the base of a brick chimney at Glasgow, Scotland, 468 ft. high, is estimated at 9 tons per sq. ft. (about 150 lbs. per sq. in.); and in heavy gales this is increased to 15 tons per sq. ft. (210 lbs. per sq. in.) on the leeward side. The leading Chicago architects allow 10 tons per sq. ft. (140 lbs. per sq. in.) on the best brick-work laid in 1 to 2 Portland cement mortar; 8 tons for good brick-work in 1 to 2 Rosendale cement mortar; and 5 tons for ordinary brick-work in lime mortar. Ordinary brick piers have been known to bear 40 tons per sq. ft. (560 lbs. per sq. in.) for several days without any sign of failure.

Tables 19 and 20 appear to show that present practice is very conservative with regard to the pressure allowed on brick masonry. According to Table 19 (page 164), the ultimate strength of the best brick laid in ordinary lime mortar is 110 tons per sq. ft.; if laid in 1 to 2 Portland cement mortar, 180 tons; and by Table 20 (page 166) the strength of ordinary brick in 1 to 2 lime mortar is 100 tons per sq. ft., and in 1 to 3 Portland cement mortar 140 tons. From the above, it would seem that reasonably good brick laid in good lime mortar should be safe under a pressure of 20 tons per sq. ft., and that the best brick in good Portland cement mortar should be safe under 30 tons per sq. ft. The nominal pressure allowed upon brick masonry depends upon the kind of materials employed; the degree of care with which it is executed; whether it is for a temporary or permanent, an important or unimportant structure; and, it may be added, the care with which the nominal maximum load is estimated.

249. Transverse Strength of Brick Masonry. Masonry is seldom employed where any strain except direct compression will come upon it, but sometimes it is subject to transverse strain. The transverse strength of brick-work depends theoretically upon the tensile strength of the brick and upon the adhesion and cohesion of the mortar, but practically the strength of the mortar determines the strength of the masonry. For example, in the case of a high wall whose upper portion is overthrown by a lateral force or pressure of any kind, the failure is due either (1) to the breaking of the adhesion in the bed-joints and of the cohesion of the side-joints, or (2) to the rupture of the mortar in the bed-joints alone. The latter method of failure, however, is improbable, since the cohesion

of cement mortars is always much greater than their adhesion (compare §§ 134 and 137); and hence, in estimating the resistance of the wall to overturning, it becomes necessary to fix values for both the cohesive and adhesive strength of the mortar at the time when the structure is first exposed to the action of the lateral force or pressure, and also to ascertain the relative areas of beds and side-joints in the assumed section of rupture. In good brick-work the aggregate area of the side-joints, in any section parallel to the beds, will amount to about one seventh of the total area of such section. Hence, when the masonry is liable to be subjected to transverse strains the adhesive strength of the mortar is more important than its cohesive strength.

The adhesion of mortar to brick or stone has already been discussed (§ 137). While the experiments uniformly show a relatively low adhesive power, it is well known that when old walls are demolished the adhesion of even common lime mortar is found to be very considerable. Although the adhesive power of mortar may be small as compared with its tensile strength, good brick masonry has a considerable transverse strength.

Eleven old English experiments,* made in England in 1837, gave an average resistance to flexure of 274 lbs. per sq. in. for brick cemented together with clear Portland cement, the minimum being 200 and the maximum 380. "Weisbach, upon a basis entirely different from that employed in deducing the preceding results, concludes that the resistance to flexure varies between 180 and 340 lbs. per sq. in."

250. Application. To illustrate the practical application of the fact that brick-work has a transverse strength, let it be required to compute the strain which may come upon a lintel, or girder used to support a brick wall over an opening.†

Let H = the height, in feet, of the wall above the opening;

H_m = the height, in feet, of the wall that produces a maximum strain on the lintel;

H_s = the height, in feet, of the masonry when it will just support itself over the opening;

t = the thickness, in feet, of the wall;

* Civil Engineer's and Architect's Jour., vol. i. pp. 30, 45, 102, 135.

† The principle of the following computations is from an editorial in *Engineering* (London), vol. xiv. pp. 44 and 72.

R = the modulus of rupture, in pounds per square inch, of the brick-work;

W = the weight, in pounds, of a cubic foot of the wall. W varies from 100 to 140 pounds, and for convenience is here assumed to be 144; the error is always on the safe side.

Consider the masonry as a beam fixed at both ends and loaded uniformly. Then, by the principles of the resistance of materials, when the masonry is just self-supporting, one twelfth of the weight of the wall above the opening *multiplied by* the span *is equal to* one sixth of the tensile strength *multiplied by* the thickness and also the square of the depth of the wall. The weight of the wall above the opening is $W S H_s t$. Hence

$$\tfrac{1}{12}(W S H_s t)\, S = \tfrac{1}{6}(144\, R)\, t H_s^2, \quad . \quad . \quad . \quad . \quad (1)$$

or

$$H_s = \frac{S^2}{2R}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Notice that the weight of the wall over any given opening increases as the height, while the resistance increases as the square of the height. The height for which the masonry is self-supporting is given by equation (2); for a height greater than H_s the masonry would be more than self-supporting; and for a height less than H_s the masonry would need extraneous support.

To find the relationship between the height of the wall that is self-supporting and the height that produces the maximum strain on the lintel, notice that, since the strength of the wall increases as H^2 and the weight as H, the net resistance of the wall increases as H. Consequently that portion of the wall which will be self-supporting can be represented by the $\frac{H_m}{H_s}$ part of the entire weight, and the part that must receive extraneous support can be represented by the $\left(1 - \frac{H_m}{H_s}\right)$ part of the entire weight. Since the weight of the wall over a given opening varies as the height, the weight to be supported by the lintel is proportional to $\left(1 - \frac{H_m}{H_s}\right) H_m$; hence the greatest strain on the lintel will occur when this expression is a maximum,—*i. e.*, when $H_m = \frac{1}{2} H_s$.

Substituting this value of H_m in the above expression for the load on the lintel, $\left(1 - \frac{H_m}{H_s}\right)H_m$, it becomes $\frac{1}{4}H_s$. This shows that the maximum load on the lintel is equal to one quarter of the weight of the self-supporting wall; or, since $H_m = \frac{1}{2}H_s$, the maximum load on the girder is equal to one half of the weight of the entire wall above the opening. Substituting this value of H_m in equation (2), we have

$$H_m = \frac{S^2}{4R}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Hence it appears that the height of the wall producing the maximum strain on the lintel will be equal to $\frac{S^2}{4R}$, and that one half of the wall will then be self-supporting and half will require extraneous support. Or, in other words, the greatest stress on a lintel due to a wall of any height will not be greater than that due to a distributed load of

$$\tfrac{1}{2}\,WH_m\,St = \tfrac{1}{2}\,W\frac{S^2}{4R}St = \text{nearly } 15\,\frac{S^2t}{R} \text{ pounds.} \quad . \quad (4)$$

251. *Examples.* To apply the above formula, assume that it is proposed to cut a 10-foot opening through an old brick wall, and that it is desirable to know whether the brick-work will be self-supporting, the wall rising 40 feet above the top of the opening. Substituting the above data in equation (2) gives

$$40 = \frac{(10)^2}{2R}; \quad \text{or} \quad R = 1.25 \text{ lbs. per sq. in.}$$

Hence, to be self-supporting across the opening, the wall must be capable of supporting a tensile strain of 1.25 pounds per square inch. It would be poor lime mortar that would not bear eight or ten times this. Notice that if the wall were only 4 feet high over the opening, instead of 40 feet, as above, the strength required would be 12.5 pounds per square inch.

For another illustration, assume that a brick wall 1 foot thick is to be built over a 10-foot opening, and that we wish to know whether a timber 10 inches deep and 12 inches wide will sustain the load. Assuming the beam as being fixed at the ends, the timber will sustain a uniformly distributed load of 10 tons with a deflection

of one twelfth of an inch. This is equivalent to the entire weight of the wall when 14 feet high. If the wall is to be carried higher than this, the girder must be supported temporarily, or time must be given for the mortar to set.

However, before the wall is 14 feet above the opening, the brick-work at the bottom will have attained some strength, and therefore the load on the girder will not be as great as above. The average strength of the brick-work will always be at least the average between the strength at the top and the bottom; that is, the average strength will always be more than half of that at the bottom. Since 10 tons is the maximum load allowed on the girder, and since the maximum load which comes upon it is half of the entire weight of the masonry above the opening,* the timber will receive its maximum load when the wall is twice 14 feet, or 28 feet, above the opening. The masonry may be run up 28 feet without necessitating any extraneous support for the lintel, provided time enough is allowed for the mortar to develop the average tensile strength found by substituting in (4) the maximum load allowed on the girder, and solving for R. Making this substitution gives

$$20000 = \frac{15\,(10)^3\,1}{R}, \text{ from which } R = 0.75 \text{ lb. per sq. in.}$$

With an average strength of 0.75 lb. per sq. in., the wall will become self-supporting when 70 feet above the opening.

252. Custom differs as to the manner of estimating the pressure on a girder due to a superincumbent mass of masonry. One extreme consists in assuming the masonry to be a fluid, and taking the load on the lintel as the weight of all the masonry above the opening. The opposite extreme consists in assuming the pressure to be the weight of the masonry included in a triangle of which the opening is the base and whose sides make 45° with this line. Both of these methods differ materially from the one discussed above; and neither is defensible. As the wall is several days in building, the masonry first laid attains considerable strength before the wall is completed; and hence, owing to the cohesion of the mortar, the final weight on the girder can not be equal to or compared with any fluid volume.

The principle involved in the second method would be applicable

* See discussion of equation (3), above.

to a wall composed wholly of perfectly smooth bricks. In a dry wall, the angle which the side lines make with the base would depend upon the bond and upon the relative length and breadth of the bricks. Assuming the boundary lines to make an angle of 45° with the base the method gives a load $\frac{3R}{S}$ times that (§ 250) which takes account of the transverse strength of the masonry, *i. e.*, the frictional and tensile resistance of the wall. If R is relatively large and S is small, this fraction will be more than unity, under which conditions the second method is safe. But if R is small and S is large, then this fraction is less than one, which shows that under these conditions the second method is unsafe.

The method of § 250 is quite simple and perfectly general. The substantial correctness of this method, illustrated in § 251, is proven by the fact that large openings are frequently cut through walls without providing any extraneous support; and also by the fact that walls are frequently supported over openings on timbers entirely inadequate to carry the load if the masonry did not have considerable strength as a beam. The discussion in § 251 also makes clear why frequently a temporary support is sufficient. After the masonry has been laid a short time, the strength of the mortar causes it to act as a beam. The discussion also shows the advantage of using cement mortar (or better, quick-setting cement mortar) when it is desired that the masonry shall early become self-supporting.

253. Measurement of Brick-work. The method of determining the quantity of brick masonry is governed by voluminous trade rules or by local customs, which are even more arbitrary than those for stone masonry (§ 224, which see).

The quantity is often computed in perches, but there is no uniformity of understanding as to the contents of a perch. It ranges from 16½ to 25 cubic feet.

Brick-work is also often measured by the square rod of exterior surface. No wall is reckoned as being less than a brick and a half in thickness (13 or 13½ inches), and if thicker the measurement is still expressed in square rods of this standard thickness. Unfortunately the dimensions adopted for a square rod are variable, the following values being more or less customary: 16½ feet square or

272¼ square feet, 18 feet square or 324 square feet, and 16½ square feet.

The volume of a brick is sometimes used as a unit in stating the contents of a wall. The contents of the wall are found by multiplying the number of cubic feet in the wall by the number of brick which it is assumed make a cubic foot; but as the dimensions of brick vary greatly (see § 62), this method is objectionable. A cubic foot is often assumed to contain 20 brick, and a cubic yard 600. The last two quantities are frequently used interchangeably, although the assumed volume of the cubic yard is *thirty* times that of the cubic foot.

Brick-work is also sometimes measured by allowing a certain number of brick to each superficial foot, the number varying with the thickness of the wall. A 4-inch wall (thickness = width of one brick) is frequently assumed to contain 7 bricks per sq. ft.; a 9-inch wall (thickness = width of two bricks), 14 bricks per sq. ft.; a 13-inch wall (thickness = width of three bricks), 21 bricks per sq. ft., etc.; the number of brick per square foot of the face of the wall being seven times the thickness of the wall in terms of the width of a brick.

254. The only relief from such arbitrary, uncertain, and indefinite customs is to specify that the masonry will be paid for by the cubic yard,—gross or net measurement, according to the structure or the preference of the engineer or architect.

In engineering the uniform custom is to measure the exact solid contents of the wall.

255. DATA FOR ESTIMATES. Number of Brick Required. Since the size of brick varies greatly (§ 62), it is impossible to state a rule which shall be equally accurate in all localities. If the brick be of standard size ($8\frac{1}{4} \times 4 \times 2\frac{1}{4}$ inches), and laid with ½- to ⅝-inch joints, a cubic yard of masonry will require about 410 brick; or a thousand brick will lay about 2½ cubic yards. If the joints are ¼- to ⅜-inch, a cubic yard of masonry will require about 495 brick; or a thousand brick will lay about 2 cubic yards. If the joints are ⅛ inch, a cubic yard of masonry will require about 545 face brick; or a thousand face brick will lay about 1.8 cubic yards.

In making estimates for the number of bricks required, an allowance must be made for breakage, and for waste in cutting brick to fit angles, etc. With good brick, in massive work this allowance

need not exceed 1 or 2 per cent.; but in buildings 3 to 5 per cent. is none too much.

256. Amount of Mortar Required. The proportion of mortar to brick will vary with the size of the brick and with the thickness of the joints. With the standard size of brick ($8\frac{1}{4} \times 4 \times 2\frac{1}{4}$ inches), a cubic yard of masonry, laid with $\frac{1}{2}$- to $\frac{5}{8}$-inch joints, will require from 0.35 to 0.40 of a cubic yard of mortar; or a thousand brick will require 0.80 to 0.90 of a cubic yard. If the joints are $\frac{1}{4}$ to $\frac{3}{8}$ inch, a cubic yard of masonry will require from 0.25 to 0.30 of a cubic yard of mortar; or a thousand brick will require from 0.45 to 0.55 of a cubic yard. If the joints are $\frac{1}{8}$ of an inch, a cubic yard of masonry will require from 0.10 to 0.15 of a cubic yard of mortar; or a thousand brick will require from 0.15 to 0.20 of a cubic yard.

With the above data, and the table on page 86, the amount of cement and sand required for a specified number of brick, or for a given number of yards of masonry, can readily be determined.

257. Labor Required. "A bricklayer, with a laborer to keep him supplied with materials, will lay on an average, in common house-walls, about 1,500 bricks per day of 10 working hours; in the neater outer faces of brick buildings, from 1,000 to 1,200; in good ordinary street fronts, from 800 to 1,000; and in the very finest lower-story faces used in street fronts, from 150 to 300 according to the number of angles, etc. In plain massive engineering work, he should average about 2,000 bricks per day, or 4 cu. yds. of masonry; and in large arches, about 1,500, or 3 cu. yds."*

In the United States Government buildings the labor per thousand, including tools, etc., is estimated at seven eighths of the wages for ten hours of mason and helper.

Table 21, opposite, † gives the actual labor, per cubic yard, required on some large and important jobs.

258. Cost. In the construction of the Cincinnati Southern R. R., during 1873–77, the brick lining of tunnels cost $8.50 per cu. yd.; brick-work in buildings, $7.00.‡ The average price paid for the brick-work in the new Croton Aqueduct tunnel, which supplies New York City with water, was, including everything, $10.14 per cu. yd.

* Trautwine's Engineer's Pocket-Book, p. 671.

† Trans. Am. Soc. of C. E.

‡ Report of the Chief Engineer, Dec. 1, 1877, Exhibit 3.

TABLE 21.

LABOR REQUIRED FOR BRICK MASONRY.

LOCATION AND DESCRIPTION OF THE MASONRY.	WORK REQUIRED, IN DAYS PER CUBIC YARD.
High Bridge Enlargement, N. Y. City— Lining wall and flat arches laid with very close joints.	0.714
Washington (D. C.) Aqueduct— Circular conduit, 9 feet in diameter with walls 12 inches thick..................................	0.439
St. Louis Water Works— Semi-circular conduit, 6 feet in diameter...........	0.364
New York City Storage Reservoir— Lining of gate-house walls and arches—rough work..	0.304

for lining, and $8.49 for backing. The mortar was composed of 1 part cement of the Rosendale type and 2 parts of sand.*

In Chicago in 1887, the price of brick laid in lime in interior walls was about $11 per thousand, equivalent to about $7 per cu. yd. The wages of masons were from 45 to 50 cents per hour, and of common labor from 20 to 25 cents per hour.

259. SPECIFICATIONS FOR BRICK MASONRY. For Buildings. There is not even a remote approach to uniformity in the specifications for the brick-work of buildings. Ordinarily the specifications for the brick masonry are very brief and incomplete. The following conform closely to ordinary construction. Of course, a higher grade of workmanship can be obtained by more stringent specifications.†

The brick in the exterior walls must be of good quality, hard-burned; fine, compact, and uniform in texture; regular in shape, and uniform in size.‡ One fourth of the brick in the interior walls may be what is known as soft or salmon brick (see 2, § 56). The brick must be thoroughly wet before being laid. The joints of the exterior walls shall be from ¼ to ⅜ inch thick.§ The joints of interior division-walls may be from ⅜ to ½ inch thick. The mortar shall be composed of 1 part of fresh, well-slaked lime and 2½ to 3 parts

* Report of the Aqueduct Commission, 1883–87, Table 4.

† For specifications for masonry for various purposes, see Appendix I.

‡ See § 57, page 37.

§ For the best work, omit this item and insert the following: *The outside walls shall be faced with the best pressed brick of uniform color, laid in colored mortar, with joints not exceeding one eighth of an inch in thickness.* Face brick are made a little larger (§ 62) than ordinary brick to compensate for the thinner joints.

of clean, sharp sand.* The lime-paste and the sand shall be thoroughly mixed before being used. The joints shall be well filled with the above mortar; no grout shall be used in the work. The bond must consist of five courses of stretchers to one of headers, and shall be so arranged as to thoroughly bind the exterior and interior portions of the wall to each other.

The contractor must furnish, set up, and take away his own scaffolding; he must build in such strips, plugs, blocks, scantling, etc., as are required for securing the wood-work; and must also assist in placing all iron-work, as beams, stairways, anchors, bed-plates, etc., connected with the brick-work.

260. For Sewers. The following are the specifications employed, in 1885, in the construction of brick sewers in Washington, D. C.:

"The best quality of whole new brick, burned hard entirely through, free from injurious cracks, with true even faces, and with a crushing strength of not less than 5,000 pounds per square inch, shall be used, and must be thoroughly wet by immersion immediately before laying. Every brick is required to be laid in full mortar joints, on bottom, sides, and ends, which for each brick is to be performed by one operation. In no case is the joint to be made by working in mortar after the brick has been laid. Every second course shall be laid with a line, and joints shall not exceed three eighths of an inch. The brick-work of the arches shall be properly bonded, and keyed as directed by the engineer. No portion of the brick-work shall be laid dry and afterwards grouted.

"The mortar shall be composed of cement and dry sand, in the proportion of 300 pounds of cement and 2 barrels of loose sand, thoroughly mixed dry, and a sufficient quantity of water afterwards added to form a rather stiff paste. It shall be used within an hour after mixing, and not at all if once set.

"The cement shall be of the best quality, freshly burned, and equal in every respect to the Round Top or Shepardstown cement, manufactured upon the formula of the engineer-commissioner of the District of Columbia, capable of being worked for twenty minutes in mortar without loss of strength, and shall be tested in such manner as the engineer may direct. After being mixed with water, allowed to set in air for twenty-four hours, and then immersed in water for six days, the tensile strength must be as follows:

Neat cement	95 lbs. per sq. in.
One part cement and one part sand	56 " " " "
" " " " two parts "	22 " " " "
" " " " three " "	12 " " " "

"The sand used shall be clean, sharp, free from loam, vegetable matter, or other dirt, and capable of giving the above results with the cement.

"The water shall be fresh and clean, free from earth, dirt, or sewerage.

* For masonry that is to be subjected to a heavy pressure, omit this item and insert the following: *The mortar must be composed of 1 part lime-paste, 1 part cement, and 2 parts of clean, sharp sand.* Or, if a heavier pressure is to be resisted, specify that some particular grade of cement mortar is to be used. (See §§ 246 and 247.)

"Tight mortar-boxes shall be provided by the contractor, and no mortar shall be made except in such boxes.

"The proportions given are intended to form a mortar in which every particle of sand shall be enveloped by the cement; and this result must be attained to the satisfaction of the engineer and under his direction. The thorough mixing and incorporation of all materials (preferably by machine labor) will be insisted upon. If by hand labor, the dry cement and sand shall be turned over with shovels by skilled workmen not less than six times before the water is added. After adding the water, the paste shall again be turned over and mixed with shovels by skilled workmen not less than three times before it is used."

261. For Arches. The specifications for the brick arch masonry on the Atchison, Topeka and Santa Fé Railroad are as follows:

"The bricks must be of the best quality of smooth, hard-burnt, paving bricks, well tempered and moulded, of the usual size, compact, well shaped, free from lime, cracks, and other imperfections, and must stand a pressure of 4,000 pounds per square inch without crushing. No bats will be allowed in the work except for making necessary closures. All bricks will be culled on the ground after delivery, and selected in strict accordance with these specifications.

"The mortar must be made of 1 measure of good hydraulic [Rosendale] cement and 2 measures of clean, sharp sand—or such other proportion as may be prescribed by the engineer—well mixed together with clean water, in clean mortar-beds constructed of boards, and must be used immediately after being mixed.

"The brick must be laid flush in cement mortar, and must be thoroughly wet when laid. All joints and beds must be thoroughly filled with mortar so as to leave no empty spaces whatever in the masonry of the walls and arches, which must be solid throughout. The thickness of mortar-joints must be as follows: In the walls and in the arch between bricks of the same ring, not less than three eighths of an inch ($\frac{3}{8}''$) nor more than one half inch ($\frac{1}{2}''$). In the arch between rings, not less than one half inch ($\frac{1}{2}''$) nor more than five eighths of an inch ($\frac{5}{8}''$). Each brick is to be driven into place by blows of a mallet. The bricks must be laid in the walls with the ordinary English bond, five stretcher courses to one header course. They must be laid in the arch in concentric rings, each longitudinal line of bricks breaking joints with the adjoining lines in the same ring and in the ring under it. No headers to be used in the arch."

262. Brick vs. Stone Masonry. Brick masonry is not much used, except in the walls of buildings, in lining tunnels, and in constructing sewers, the general opinion being that brick-work is in every way inferior to stone masonry. This belief may have been well founded when brick was made wholly by hand, by inexpert operatives, and imperfectly burned in the old-time kilns, the prod-

uct being then generally poor; but things have changed, and since the manufacture of brick has become a business conducted on a large scale by enterprising men, with the aid of a variety of machines and improved kilns, the product is more regular in size and quality and stronger than formerly. Brick is rapidly displacing stone for the largest and best buildings in the cities, particularly in Chicago and St. Petersburg, where the vicissitudes of the climate try masonry very severely. There are many engineering structures in which brick could be profitably employed instead of stone; as, for example, the walls of box-culverts, cattle-guards, etc., and the less important bridge piers and abutments, particularly of highway bridges.

Brick-work is superior to stone masonry in several respects, as follows: 1. In many localities brick is cheaper than stone, since the former can be made near by while the latter must be shipped. 2. As brick can be laid by less skillful masons than stone, it costs less to lay it. 3. Brick is more easily handled than stone, and can be laid without any hoisting apparatus. 4. Brick requires less fitting at corners and openings. 5. Brick masonry is less liable to great weakness through inaccurate dressing or bedding. 6. Brick-work resists fire better than limestone, granite, or marble, sandstone being the only variety of stone that can compare with brick in this respect. 7. Good brick stands the effect of weathering and of the acids in the atmosphere better than sandstones, and in durability even approaches some of the harder stones (see §§ 31, 32). 8. All masonry fails when the mortar in its joints disintegrates or becomes dislodged; therefore brick masonry will endure the vicissitudes of the weather as well as stone masonry, or even better, since the former usually has thinner joints.

Brick-work is not as strong as ashlar masonry, but costs less; while it is stronger and costs more than ordinary rubble.

263. Brick Masonry Impervious to Water. It sometimes becomes necessary to prevent the percolation of water through brick walls. A cheap and effective process has not yet been discovered, and many expensive trials have proved failures. The following account* gives the details of two experiments that were entirely successful.

"The face walls of the back bays of the gate-houses of the new

* Abstract of a paper by Wm. L. Dearborn, in Trans. Am. Soc. of C. E., vol. i. pp. 203-8.

Croton reservoir, located north of Eighty-sixth Street, in Central Park, New York City, were built of the best quality of hard-burnt brick, laid in mortar composed of hydraulic cement of New York [Ulster Co. Rosendale] and sand mixed in the proportion of one measure of cement to two of sand. The space between the walls was 4 feet, and was filled with concrete. The face walls were laid up with great care, and every precaution was taken to have the joints well filled and to insure good work. The walls are 12 inches thick and 40 feet high; and the bays, when full, generally have 36 feet of water in them.

"When the reservoir was first filled and the water let into the gate-houses, it was found to filter through these walls to a considerable amount. As soon as this was discovered the water was drawn out of the bays, with the intention of attempting to remedy or prevent this infiltration. After carefully considering several modes of accomplishing the object desired, I [Dearborn] came to the conclusion to try 'Sylvester's Process for Repelling Moisture from External Walls.'

"The process consists in using two washes or solutions for covering the surface of the walls—one composed of Castile soap and water, and one of alum and water. The proportions are three quarters of a pound of soap to one gallon of water, and half a pound of alum to four gallons of water, both substances to be perfectly dissolved in water before being used. The walls should be perfectly clean and dry, and the temperature of the air not above 50° Fahr. when the compositions are applied.

"The first, or soap-wash, should be laid on, when boiling hot, with a flat brush, taking care to form a froth on the brick-work. This wash should remain 24 hours, so as to become dry and hard before the second, or alum, wash is applied, which should be done in the same manner as the first. The temperature of this wash, when applied, may be 60° or 70° Fahr.; and this also should remain 24 hours before a second coat of the soap-wash is put on. These coats are to be applied alternately until the walls are made impervious to water. The alum and soap thus combined form an insoluble compound, filling the pores of the masonry and entirely preventing the water from entering the walls.

"Before applying these compositions to the walls of the bays some experiments were made to test the absorption of water by

bricks under pressure after being covered with these washes, in order to determine how many coats the walls would require to render them impervious to water. To do this, a strong wooden box large enough to hold two bricks was made, put together with screws, and in the top was inserted a 1-inch pipe 40 feet long. In this box were placed two bricks, after being made perfectly dry, which were then covered with a coat of each of the washes, as before directed, and weighed. They were then subjected to a column of water 40 feet high; and after remaining a sufficient length of time they were taken out and weighed again, to ascertain the amount of water they had absorbed. The bricks were then dried, and again coated with the washes and weighed, and subjected to pressure as before, this operation being repeated until the bricks were found not to absorb any water. Four coatings rendered the bricks impenetrable under the pressure of a 40-foot head. The mean weight of the bricks (dry) before being coated was 3¾ lbs.; the mean absorption was one half-pound of water. A hydrometer was used in testing the solutions.

"As this experiment was made in the fall and winter (1863), after the temporary roofs were put on to the gate-house, artificial heat had to be resorted to to dry the walls and keep the air at a proper temperature. The cost was 10 cents per sq. ft. As soon as the last coat had become hard, the water was let into the bays, and the walls were found to be perfectly impervious to water, and they remain so in 1870, after about 6½ years.

264. "The brick arch of the footway of High Bridge is the arc of a circle, 29½ feet radius, and is 12 inches thick; the width on top is 17 feet, and the length covered is 1,381 feet. The first two courses of the brick of the arch are composed of the best hard-burnt brick, laid edgewise in mortar composed of 1 part, by measure, of hydraulic cement of New York [Ulster Co. Rosendale] and 2 parts of sand. The top of these bricks, and the inside of the granite coping against which the two top courses of brick rest, was covered, when perfectly dry, with a coat of asphalt one half an inch thick, laid on when the asphalt was heated to a temperature of from 360° to 518° Fahr. On top of this was laid a course of brick flatwise, dipped in asphalt, and laid when the asphalt was hot; and the joints were run full of hot asphalt. On top of this, a course of pressed brick was laid flatwise in hydraulic cement mortar, forming the paving and floor of the bridge.

"The area of the bridge covered with asphalted brick was 23,065 sq. ft. There were used 94,200 lbs. of asphalt, 33 barrels of coal tar, 10 cu. yds. of sand, and 93,800 bricks. The asphalt was the Trinidad variety; and was mixed with 10 per cent., by measure, of coal tar, and 25 per cent. of sand. The time occupied was 109 days of masons, and 148 days of laborers. Two masons and two laborers will melt and spread, of the first coat, 1,650 sq. ft. per day. The total cost of this coat was 5¼ cents per sq. ft., exclusive of duty on asphalt.

"There were three grooves, 2 inches wide by 4 inches deep, made entirely across the brick arch immediately under the first coat of asphalt, thus dividing the arch into four equal parts. The grooves were filled with elastic paint cement. This arrangement was intended to guard against the evil effects of the contraction of the arch in winter; for, since it was expected to yield slightly at these points and at no other, the elastic cement would prevent any leakage there. The entire experiment has proved a very successful one, and the bridge has remained perfectly tight.

"In proposing the above plan for working the asphalt with the brick-work, the object was to avoid depending on a large continuous surface of asphalt, as is usual in covering arches, which very frequently cracks from the greater contraction of the asphalt than that of the masonry with which it is in contact, the extent of the asphalt on this work being only about one quarter of an inch to each brick. This is deemed to be an essential element in the success of the impervious covering."

265. Efflorescence. Masonry, particularly in moist climate or in damp places—as cellar walls,—is frequently disfigured by the formation of a white efflorescence on the surface. This deposit generally originates with the mortar, but frequently spreads over the entire face of the wall. The water which is absorbed by the mortar dissolves the salts of soda, potash, magnesia, etc., contained in the lime or cement, and on evaporating deposits these salts as a white efflorescence on the surface. With lime mortar the deposit is frequently very heavy, particularly on plastering; and, usually, it is heavier with Rosendale than with Portland cement. The efflorescence sometimes originates in the brick, particularly if the brick was burned with sulphurous coal, or was made from clay containing iron-pyrites; and when the brick gets wet, the water dissolves

the sulphates of lime and magnesia, and on evaporating leaves the crystals of these salts on the surface. Frequently the efflorescence on the brick is due to the absorption by the brick of the impregnated water from the mortar.

This efflorescence is objectionable because of the unsightly appearance which it often produces, and also because the crystallization of these salts within the pores of the mortar and of the brick or stone causes disintegration which is in many respects like frost.

As a preventive, Gillmore recommends* the addition of 100 lbs. of quicklime and 8 to 12 lbs. of any cheap animal fat to each barrel of cement. The lime is simply a vehicle for the fat, which should be thoroughly incorporated with the lime before slaking. The object of the fat is to saponify the alkaline salts. The method is not entirely satisfactory, since the deposit is only made less prominent and less effective, and not entirely removed or prevented.

The efflorescence may be entirely prevented, whatever its origin, by applying Sylvester's washes (see § 263) to the entire external surfaces of the wall; and, since usually the efflorescence is due to the water absorbed by the mortar, it can generally be prevented, and can always be much diminished, by using mortar which is itself impervious to water (see § 141). The latter is the cheaper method, particularly if the impervious mortar be used only for the face of the joints. If the wall stands in damp ground, one or more of the horizontal joints just above the surface should be laid in impervious mortar, or better, the brick for several courses should be rendered impervious and be laid in impervious mortar to prevent the wall's absorbing moisture from below.

* "Limes, Hydraulic Cements, and Mortars," p. 206.

PART III.

FOUNDATIONS.

CHAPTER IX.

INTRODUCTORY.

266. Definitions. The term foundation is ordinarily used indifferently for either the lower courses of a structure of masonry or the artificial arrangement, whatever its character, on which these courses rest. For greater clearness, the term *foundation* will here be restricted to the artificial arrangement, whether timber or masonry, which supports the main structure; and the prepared surface upon which this artificial structure rests will be called the *bed of the foundation.* There are many cases in which this distinction can not be adhered to strictly.

267. Importance of the Subject. The foundation, whether for the more important buildings or for bridges and culverts, is the most critical part of a masonry structure. The failures of works of masonry due to faulty workmanship or to an insufficient thickness of the walls are rare in comparison with those due to defective foundations. When it is necessary, as so frequently it is at the present day, to erect gigantic edifices—as high buildings or long-span bridges—on weak and treacherous soils, the highest constructive skill is required to supplement the weakness of the natural foundation by such artificial preparations as will enable it to sustain such massive and costly burdens with safety.

Probably no branch of the engineer's art requires more ability and skill than the construction of foundations. The conditions governing safety are generally capable of being calculated with as much practical accuracy in this as in any other part of a con-

struction; but, unfortunately, practice is frequently based upon empirical rules rather than upon a scientific application of fundamental principles. It is unpardonable that any liability to danger or loss should exist from the imperfect comprehension of a subject of such vital importance. Ability is required in determining the conditions of stability; and greater skill is required in fulfilling these conditions, that the cost of the foundation may not be proportionally too great. The safety of a structure may be imperiled, or its cost unduly increased, according as its foundations are laid with insufficient stability, or with provision for security greatly in excess of the requirements. The decision as to what general method of procedure will probably be best in any particular case is a question that can be decided with reasonable certainty only after long experience in this branch of engineering; and after having decided upon the general method to be followed, there is room for the exercise of great skill in the means employed to secure the desired end. The experienced engineer, even with all the information which he can derive from the works of others, finds occasion for the use of all his knowledge and best common sense.

The determination of the conditions necessary for stability can be reduced to the application of a few fundamental principles which may be studied from a text-book; but the knowledge required to determine beforehand the method of construction best suited to the case in hand, together with its probable cost, comes only by personal experience and a careful study of the experiences of others. The object of Part III. is to classify the principles employed in constructing foundations, and to give such brief accounts of actual practice as will illustrate the applications of these principles.

268. Plan of Proposed Discussion. In a general way, soils may be divided into three classes: (1) ordinary soils, or those which are capable, either in their normal condition or after that condition has been modified by artificial means, of sustaining the load that is to be brought upon them; (2) compressible soils, or those that are incapable of directly supporting the given pressure with any reasonable area of foundation; and (3) semi-liquid soils, or those in which the fluidity is so great that they are incapable of supporting any considerable load. Each of the above classes gives rise to a special method of constructing a foundation.

1. With a soil of the first class, the bearing power may be in-

creased by compacting the surface or by drainage; or the area of the foundation may be increased by the use of masonry footing courses, inverted masonry arches, or one or more layers of timbers, railroad rails, iron beams, etc. Some one of these methods is ordinarily employed in constructing foundations on land; as, for example, for buildings, bridge abutments, sewers, etc. Usually all of these methods are inapplicable to bridge piers, *i. e.*, for foundations under water, owing to the scouring action of the current and also to the obstruction of the channel by the greatly extended base of the foundation.

2. With compressible soils, the area of contact may be increased by supporting the structure upon piles of wood or iron, which are sustained by the friction of the soil on their sides and by the direct pressure on the soil beneath their bases. This method is frequently employed for both buildings and bridges.

3. A semi-fluid soil must generally be removed entirely and the structure founded upon a lower and more stable stratum. This method is specially applicable to foundations for bridge piers.

There are many cases to which the above classification is not strictly applicable.

For convenience in study, the construction of foundations will be discussed, in the three succeeding chapters, under the heads *Ordinary Foundations*, *Pile Foundations*, and *Foundations under Water*. However, the methods employed in each class are not entirely distinct from those used in the others.

CHAPTER X.

ORDINARY FOUNDATIONS.

269. In this chapter will be discussed the method of constructing the foundations for buildings, bridge abutments, culverts, or, in general, for any structure founded upon dry, or nearly dry, ground. This class of foundations could appropriately be called Foundations for Buildings, since these are the most numerous of the class.

This chapter is divided into three articles. The first treats of the soil, and includes (*a*) the methods of examining the site to determine the nature of the soil, (*b*) a discussion of the bearing power of different soils, and (*c*) the methods of increasing the bearing power of the soil. The second article treats of the method of designing the footing courses, and includes (*a*) the method of determining the load to be supported, and (*b*) the method of increasing the area of the foundation. The third contains a few remarks concerning the practical work of laying the foundation.

Art. 1. The Soil.

270. Examination of the Site. The nature of the soil to be built upon is evidently the first subject for consideration, and if it has not already been revealed to a considerable depth, by excavations for buildings, wells, etc., it will be necessary to make an examination of the subsoil preparatory to deciding upon the details of the foundation. It will usually be sufficient, after having dug the foundation pits or trenches, to examine the soil with an iron rod or a post-auger from 3 to 5 feet further, the depth depending upon the nature of the soil, and the weight and importance of the intended structure.

In soft soil, soundings 40 or 50 feet deep can be made by driving a small (say $\frac{3}{4}$-inch) gas-pipe with a hammer or maul from a temporary scaffold, the height of which will of course depend upon the length of the sections of the pipe. If samples of the soil are desired,

use a 2-inch pipe open at the lower end. If much of this kind of work is to be done, it is advisable to fit up a hand pile-driving machine (see § 335), using a block of wood for the dropping weight. Borings 50 to 100 feet deep can be made very expeditiously in common soil or clay with a common wood-auger turned by men, with levers 2 or 3 feet long. The auger will bring up samples sufficient to determine the nature of the soil, but not its compactness, since it will probably be compressed somewhat in being cut off.

When the testing must be made through sand or loose soil, it may be necessary to drive down an iron tube to prevent the soil from falling into the hole. The sand may be removed from the inside of the tube with an auger, or with the "sand-pump" used in digging artesian wells. When the subsoil is composed of various strata and the structure demands extraordinary precaution, borings must be made with the tools employed for boring artesian wells.*

271. If the builder desires to avoid, on the one hand, the unnecessarily costly foundations which are frequently constructed, or, on the other hand, those insufficient foundations evidences of which are often seen, it may be necessary, after opening the trenches, to determine the supporting power of the soil by applying a test load.

In the case of the capitol at Albany, N. Y., the soil was tested by applying a measured load to a square foot and also to a square yard. The machine used was a mast of timber 12 inches square, held vertical by guys, with a cross-frame to hold the weights. For the smaller area, a hole 3 feet deep was dug in the blue clay at the bottom of the foundation, the hole being 18 inches square at the top and 14 inches at the bottom. Small stakes were driven into the ground in lines radiating from the center of the hole, the tops being brought exactly to the same level; then any change in the surface of the ground adjacent to the hole could readily be detected and measured by means of a straight-edge. The foot of the mast was placed in the hole, and weights applied. No change in the surface of the adjacent ground was observed until the load reached 5.9 tons per sq. ft., when an uplift of the surrounding earth was noted in the form of a ring with an irregularly rounded surface, the contents of which, above the previous surface, measured 0.09 cubic feet. Similar experiments were made by applying the load to

* For illustrations of tools for this purpose, see Engineering News, vol. 21, p.324.

a square yard with essentially the same results. The several loads were allowed to remain for some time, and the settlements observed.*

Similar experiments were made in connection with the construction of the Congressional Library Building, Washington, D. C., with a frame which rested upon 4 foot-plates each a foot square. The frame could be moved from place to place on wheels, and the test was applied at a number of places.

272. Bearing Power of Soils. It is scarcely necessary to say that soils vary greatly in their bearing power, ranging as they do from the condition of hardest rock, through all intermediate stages, to a soft or semi-liquid condition, as mud, silt, or marsh. The best method of determining the load which a specific soil will bear is by direct experiment (§ 271); but good judgment and experience, aided by a careful study of the nature of the soil—its compactness and the amount of water contained in it—will enable one to determine, with reasonable accuracy, its probable supporting power. The following data are given to assist in forming an estimate of the load which may safely be imposed upon different soils.

273. Rock. The ultimate crushing strength of stone, as determined by crushing small *cubes*, ranges from 180 tons per square foot for the softest stone—such as are easily worn by running water or exposure to the weather—to 1,800 tons per square foot for the hardest stones (see page 10). The crushing strength of slabs, *i. e.*, of prisms of a less height than width, increases as the height decreases. A prism one half as high as wide is about twice as strong as a cube of the same material. If a slab be conceived as being made up of a number of cubes placed side by side, it is then easy to see why the slab is stronger than a cube. The exterior cubes prevent the detachment of the disk-like pieces (Fig. 1) from the sides of the interior cubes; and hence the latter are greatly strengthened, which materially increases the strength of the slab. In testing cubes and slabs the pressure is applied uniformly over the entire upper surface of the test specimen; and, reasoning from analogy, it seems probable that when the pressure is applied to only a small part of the surface, as in the case of foundations on rock, the strength will be much greater than that of cubes of the same material.

The table on page 190 contains the results of experiments made

* W. J. McAlpine, the engineer in charge, in Trans. Am. Soc. C. E., vol. ii. p. 287.

by the author, and shows conclusively that a unit of material has a much greater power of resistance when it forms a portion of a larger mass than when isolated in the manner customary in making experiments on crushing strength.

The ordinary "crushing strength" given in next to the last column of Table 22 was obtained by crushing cubes of the identical materials employed in the other experiments. The concentrated pressure was applied by means of a hardened steel die thirty-eight sixty-fourths of an inch in diameter (area = 0.277 sq. in.). All the tests were made between self-adjusting parallel plates of a hydrostatic testing-machine. No packing was used in either series of experiments; that is to say, the pressed surfaces were the same in both series. However, the block of limestone 7 inches thick (Experiments Nos. 8 and 13) is an exception in this respect. This block had been sawed out and was slightly hollow, and it was thought not to be worth while to dress it down to a plane. As predicted before making the test, the block split each time in the direction of the hollow. If the bed had been flat, the block would doubtless have shown a greater strength. The concentrated pressure was generally applied near the corner of a large block, and hence the distance from the center of the die to the edge of the block is to the nearest edge. Frequently the block had a ragged edge, and therefore these distances are only approximate. The quantity in the last column—"Ratio"—is the crushing load per square inch for concentrated pressures *divided by* the crushing load per square inch for uniform pressure.

The experiments are tabulated in an order intended to show that the strength under concentrated pressure varies (1) with the thickness of the block and (2) with the distance between the die and the edge of the material being tested. It is clear that the strength increases very rapidly with both the thickness and the distance from the edge to the point where the pressure is applied. Therefore we conclude that the compressive strength of cubes of a stone gives little or no idea of the ultimate resistance of the same material when in thick and extensive layers in its native bed.

274. The safe bearing power of rock is certainly *not less* than one tenth of the ultimate crushing strength of *cubes;* that is to say, the safe bearing power of solid rock is *not less* than 18 tons per sq. ft. for the softest rock and 180 for the strongest. It is safe to say

TABLE 22.

COMPRESSIVE STRENGTH WHEN THE PRESSURE IS APPLIED ON ONLY A PART OF THE UPPER SURFACE.

Reference No.	Material.	Thickness of Block.	Center of Die from Edge.	No. of Trials.	Crushing Strength per Square Inch —Concentrated Pressure.	No. of Trials.	Crushing Strength per Square Inch—Distributed Pressure.	Ratio.
1	Lime Mortar......	¼ in.	2 in.	4	3,610	3	1,340	2.7
2	Marble............	1 "	2 "	4	18,050	3	10,500	1.7
3	"	2 "	2 "	3	36,100	3	10,100	3.6
4	Brick..............	2¼ "	2 "	11	11,801	13	2,654	5.1
5	Limestone........	3 "	2 "	4	31,046	3	3,453	9.0
6	Sandstone.........	3 "	2 "	2	51,600	3	3,696	14.0
7	Limestone........	4 "	2 "	3	75,361	2	4,671	16.0
8	"	7 "	2 "	2	64,077	5	3,453	18.5
6	Sandstone.........	3 "	2 "	2	51,600	3	3,696	14.0
9	"	3 "	3 "	1	59,204	"	"	16.0
10	"	3 "	4 "	1	75,810	"	"	20.5
7	Limestone........	4 "	2 "	3	75,361	2	4,761	16.0
11	"	4 "	3 "	3	102,900	"	"	22.0
12	"	4 "	4 "	1	111,188	"	"	24.0
8	"	7 "	2 "	2	64,077	5	3,453	18.5
13	"	7 "	4 "	1	87,720	"	"	25.0
14	Clay, which for years has safely carried, without appreciable settlement, buildings concentrating 1½ to 2 tons per square foot (20 to 28 pounds per square inch), when tested in the form of cubes was crushed with 4 to 8 pounds per square inch. In this case the average "ratio" is 4.3.							

that almost any rock, from the hardness of granite to that of a soft crumbling stone easily worn by exposure to the weather or to running water, when well bedded will bear the heaviest load that can be brought upon it by any masonry construction.

It scarcely ever occurs in practice that rock is loaded with the full amount of weight which it is capable of sustaining, as the extent of base necessary for the stability of the structure is generally sufficient to prevent any undue pressure coming on the rock beneath.

275. Clay. The clay soils vary from slate or shale, which will support any load that can come upon it, to a soft, damp clay which will squeeze out in every direction when a moderately heavy pres-

sure is brought upon it. Foundations on clay should be laid at such depths as to be unaffected by the weather; since clay, at even considerable depths, will gain and lose considerable water as the seasons change. The bearing power of clayey soils can be very much improved by drainage (§ 285), or by preventing the penetration of water. If the foundation is laid upon undrained clay, care must be taken that excavations made in the immediate vicinity do not allow the clay under pressure to escape by oozing away from under the building. When the clay occurs in strata not horizontal, great care is necessary to prevent this flow of the soil. When coarse sand or gravel is mixed with the clay, its supporting power is greatly increased, being greater in proportion as the quantity of these materials is greater. When they are present to such an extent that the clay is just sufficient to bind them together, the combination will bear as heavy loads as the softer rocks.

276. The following data on the bearing power of clay will be of assistance in deciding upon the load that may safely be imposed upon any particular clayey soil. From the experiments made in connection with the construction of the capitol at Albany, N. Y., as described in § 271, the conclusion was drawn that the extreme supporting power of that soil was less than 6 tons per sq. ft., and that the load which might be safely imposed upon it was 2 tons per sq. ft. "The soil was blue clay containing from 60 to 90 per cent. of alumina, the remainder being fine siliceous sand. The soil contains from 27 to 43, usually about 40, per cent. of water; and various samples of it weighed from 81 to 101 lbs. per cu. ft." In the case of the Congressional Library (§ 271), the ultimate supporting power of "yellow clay mixed with sand" was 13½ tons per sq. ft.; and the safe load was assumed to be 2½ tons per sq. ft. Experiments made on the clay under the piers of the bridge across the Missouri at Bismarck, with surfaces 1¼ inches square, gave an average ultimate bearing power of 15 tons per sq. ft.*

The stiffer varieties of what is ordinarily called clay, when kept dry, will safely bear from 4 to 6 tons per sq. ft.; but the same clay, if allowed to become saturated with water, can not be trusted to bear more than 2 tons per sq. ft. At Chicago, the load ordinarily put on a thin layer of clay (hard above and soft below, resting on a

* Report of the engineer, Geo. S. Morison.

thick stratum of quicksand) is 1½ to 2 tons per sq. ft.; and the settlement, which usually reaches a maximum in a year, is about 1 inch per ton of load. Experience in central Illinois shows that, if the foundation is carried down below the action of frost, the clay subsoil will bear 1½ to 2 tons per sq. ft. without appreciable settling. Rankine gives the safe load for compressible soils as 1¼ to 1⅝ tons per sq. ft.

277. Sand. The sandy soils vary from coarse gravel to fine sand. The former when of sufficient thickness forms one of the firmest and best foundations; and the latter when saturated with water is practically a liquid. Sand when dry, or wet sand when prevented from spreading laterally, forms one of the best beds for a foundation. Porous, sandy soils are, as a rule, unaffected by stagnant water, but are easily removed by running water; in the former case they present no difficulty, but in the latter they require extreme care at the hands of the constructor, as will be considered later.

278. Compact gravel or clean sand, in beds of considerable thickness, protected from being carried away by water, may be loaded with 8 to 10 tons per sq. ft. with safety. In an experiment in France, clean river-sand compacted in a trench supported 100 tons per sq. ft. Sand well cemented with clay and compacted, if protected from water, will safely carry 4 to 6 tons per sq. ft.

The piers of the Cincinnati Suspension Bridge are founded on a bed of coarse gravel 12 feet below low-water, although solid limestone was only 12 feet deeper; if the friction on the sides of the pier* be disregarded, the maximum pressure on the gravel is 4 tons per sq. ft. The piers of the Brooklyn Suspension Bridge are founded 44 feet below the bed of the river, upon a layer of sand 2 feet thick resting upon bed-rock; the maximum pressure is about 5½ tons per sq. ft.

At Chicago sand and gravel about 15 feet below the surface are successfully loaded with 2 to 2½ tons per sq. ft. At Berlin the safe load for sandy soil is generally taken at 2 to 2½ tons per sq. ft. The Washington Monument, Washington, D. C., rests upon a bed of *very* fine sand two feet thick underlying a bed of gravel and bowlders; the ordinary pressure on certain parts of the foundation is not far from 11 tons per sq. ft., which the wind may increase to nearly 14 tons per sq. ft.

* For the amount of such friction, see §§ 418–19 and § 455.

279. Semi-Liquid Soils. With a soil of this class, as mud, silt, or quicksand, it is customary (1) to remove it entirely, or (2) to sink piles, tubes, or caissons through it to a solid substratum, or (3) to consolidate the soil by adding sand, earth, stone, etc. The method of performing these operations will be described later. Soils of a soft or semi-liquid character should never be relied upon for a foundation when anything better can be obtained; but a heavy superstructure may be supported by the upward pressure of a semi-liquid soil, in the same way that water bears up a floating body.

According to Rankine,* a building will be supported when the pressure at its base is $w\,h\left(\frac{1 + \sin \alpha}{1 - \sin \alpha}\right)^2$ per unit of area, in which expression w is the weight of a unit volume of the soil, h is the depth of immersion, and α is the angle of repose of the soil. If $\alpha = 5°$, then according to the preceding relation the supporting power of the soil is 1.4 $w\,h$ per unit of area; if $\alpha = 10°$, it is 2.0 $w\,h$; and if $\alpha = 15°$, it is 2.9 $w\,h$. The weight of soils of this class, *i. e.*, mud, silt, and quicksand, varies from 100 to 130 lbs. per cu. ft. Rankine gives this formula as being applicable to any soil; but since it takes no account of cohesion, for most soils it is only roughly approximate, and gives results too small. The following experiment seems to show that the error is considerable. "A 10-foot square base of concrete resting on mud, whose angle of repose was 5 to 1 [$\alpha = 11\frac{1}{3}°$], bore 700 lbs. per sq. ft."† This is $2\frac{1}{4}$ times the result by the above formula, using the maximum value of w.

Large buildings have been securely founded on quicksand by making the base of the immersed part as large and at the same time as light as possible. Timber in successive layers (§ 309) or grillage on piles (§ 320) is generally used in such cases. This class of foundations is frequently required in constructing sewers in water-bearing sands, and though apparently presenting no difficulties, such foundations often demand great skill and ability.

280. It is difficult to give results of the safe bearing power of soils of this class. A considerable part of the supporting power is derived from the friction on the vertical sides of the foundation; hence the bearing power depends in part upon the area of the side surface in contact with the soil. Furthermore, it is difficult to de-

* See Rankine's Civil Engineering, p. 379.

† Proc. Inst. of C. E., vol. xviii. p. 498.

termine the exact supporting power of a plastic soil, since a considerable settlement is certain to take place with the lapse of time. The experience at New Orleans with alluvial soil and a few experiments* that have been made on quicksand seem to indicate that with a load of ½ to 1 ton per square foot the settlement will not be excessive.

281. Bearing Power: Summary. Gathering together the results of the preceding discussion, we have the following table:

TABLE 23.

SAFE BEARING POWER OF SOILS.

KIND OF MATERIAL.	SAFE BEARING POWER IN TONS PER SQ. FT.	
	Min.	Max.
Rock—the hardest—in thick layers, in native bed (§ 274)	200	—
" equal to best ashlar masonry (§ 274)............	25	30
" " " " brick " "	15	20
" " " poor " " "	5	10
Clay, in thick beds, always dry (§ 276)................	4	6
" " " " moderately dry (§ 276)...........	2	4
" soft (§ 276)................................	1	2
Gravel and coarse sand, well cemented (§ 278)..........	8	10
Sand, compact and well cemented, "	4	6
" clean, dry................ "	2	4
Quicksand, alluvial soils, etc. (§ 280)................	0.5	1

282. Conclusion. It is well to notice that there are some practical considerations that modify the pressure which may safely be put upon a soil. For example, the pressure on the foundation of a tall chimney should be considerably less than that of the low massive foundation of a fire-proof vault. In the former case a slight inequality of bearing power, and consequent unequal settling, might endanger the stability of the structure; while in the latter no serious harm would result. The pressure per unit of area should be less for a light structure subject to the passage of heavy loads—as,

* Trans. Am. Soc. of C. E., vol. xiv. p. 182; *Engineering*, vol. xx. p. 103; Proc. Inst. of C. E., vol. xvii. p. 443; Cleeman's Railroad Practice, pp. 103–4.

for example, a railroad viaduct—than for a heavy structure subject only to a quiescent load, since the shock and jar of the moving load are far more serious than the heavier quiescent load.

The determination of the safe bearing power of soils, particularly when dealing with those of a semi-liquid character, is not the only question that must receive careful attention. In the foundations for buildings, it may be necessary to provide a safeguard against the soil's escaping by being pressed out laterally into excavations in the vicinity. In the foundations for bridge abutments, it may be necessary to consider what the effect will be if the soil around the abutment becomes thoroughly saturated with water, as it may during a flood; or what the effect will be if the soil is deprived of its lateral support by the washing away of the soil adjacent to the abutment. The provision to prevent the wash and undermining action of the stream is often a very considerable part of the cost of the structure. The prevention of either of these liabilities is a problem by itself, to the solution of which any general discussion will contribute but little.

283. Improving the Bearing Power of the Soil. When the soil directly under a proposed structure is incapable, in its normal state, of sustaining the load that will be brought upon it, the bearing power may be increased (1) by increasing the depth of the foundation, (2) by draining the site, (3) by compacting the soil, or (4) by adding a layer of sand.

284. Increasing the Depth. The simplest method of increasing the bearing power is to dig deeper. Ordinary soils will bear more weight the greater the depth reached, owing to their becoming more condensed from the superincumbent weight. Depth is especially important with clay, since it is then less liable to be displaced laterally owing to other excavations in the immediate vicinity, and also because at greater depths the amount of moisture in it will not vary so much.

In any soil, the bed of the foundation should be below the reach of frost. Even a foundation on bed-rock should be below the frost line, else water may get under the foundation through fissures, and, freezing, do damage.

285. Drainage. Another simple method of increasing the bearing power of a soil is to drain it. The water may find its way to the bed of the foundation down the side of the wall, or by percola-

tion through the soil, or through a seam of sand. In most cases the bed can be sufficiently drained by covering it with a layer of gravel—the thickness depending upon the plasticity of the soil,—and then surrounding the building with a tile-drain laid a little below the foundation. In extreme cases, it is necessary to enclose the entire site with a puddle-wall to cut off drainage water from a higher area.

286. Springs. In laying foundations, springs are often met with, and sometimes prove very troublesome. The water may be excluded from the foundation pit by driving sheet piles, or by plugging the spring with concrete. If the flow is so strong as to wash the cement out before it has set, a heavy canvas covered with pitch, etc., upon which the concrete is deposited, is sometimes used; or the water may be carried away in temporary channels, until the concrete in the artificial bed shall have set, when the water-ways may be filled with semi-fluid cement mortar. Below is an account of the method of stopping a very troublesome spring encountered in laying the foundation of the dry-dock at the Brooklyn Navy Yard.

"The dock is a basin composed of stone masonry resting on piles. The foundation is 42 feet below the surface of the ground and 37 feet below mean tide. In digging the pit for the foundation, springs of fresh water were discovered near the bottom, which proved to be very troublesome. The upward pressure of the water was so great as to raise the foundation, however heavily it was loaded. The first indication of undermining by these springs was the settling of the piles of the dock near by. In a day it made a cavity in which a pole was run down 20 feet below the foundation timbers. Into this hole were thrown 150 cubic feet of stone, which settled 10 feet during the night; and 50 cubic feet more, thrown in the following day, drove the spring to another place, where it burst through a bed of concrete 2 feet thick. This new cavity was filled with concrete, but the precaution was taken of putting in a tube so as to permit the water to escape; still it burst through, and the operation was repeated several times, until it finally broke out through a heavy body of cement 14 feet distant. In this place it undermined the foundation piles. These were then driven deeper by means of followers; and a space of 1,000 square feet around the spring was then planked, forming a floor on which was laid a layer of brick in

dry cement, and on that a layer of brick set in mortar, and the foundation was completed over all. Several vent-holes were left through the floor and the foundation for the escape of the water. The work was completed in 1851, and has stood well ever since." *

287. Consolidating the Soil. A soft, clayey soil can be greatly improved by spreading a thin layer of sand, dry earth, or broken stone over the bed of the foundation and pounding it into the soil. If the soil is very soft, compacting the surface will be insufficient; in this case the soil may be consolidated to a considerable depth by driving short piles into it. For this purpose small piles—say 6 feet long and 6 inches in diameter—serve better than large ones; and they can be driven with a hand-maul or by dropping a heavy block of wood with a tackle attached to any simple frame, or by a hand pile-driver (§ 335). They may be driven as close together as necessary, although 2 to 4 feet in the clear is usually sufficient. The latter method of compacting the soil is far more efficient than pounding the surface. In the case of impact upon earth, the immediate layers are compressed at once, and by their inertia and adhesion to the surrounding soil they intercept the effect of the blow, and thus prevent the consolidation of the lower strata. Even though the effect of a blow is not communicated to any considerable depth, the heavy masses of masonry make themselves felt at great depth, and hence for heavy buildings it is necessary to consolidate the lower strata. This can be done most easily and most efficiently by driving piles (see Art. 2).

In this connection it is necessary to remember that clay is compressible, while sand is not. Hence this method of consolidating soils is not applicable to sand, and is not very efficient in soils largely composed of it.

288. Sand Piles. Experiments show that in compacting the soil by driving piles, it is better to withdraw them and *immediately* fill the holes with sand, than to allow the wooden piles to remain. This advantage is independent of the question of the durability of the wood. When the wooden pile is driven, it compresses the soil an amount nearly or quite equal to the volume of the pile, and when the latter is withdrawn this consolidation remains, at least temporarily. If the hole is immediately filled with sand this com-

* Delafield's Foundations in Compressible Soils, p. 14—a pamphlet published by the Engineer's Department of the U. S. Army.

pression is retained permanently, and the consolidation may be still farther increased by ramming the sand in in thin layers, owing to the ability of the latter to transmit pressure laterally. And further, the sand pile will support a greater load than the wooden pile; for, since the sand acts like innumerable small arches reaching from one side of the hole to the other, more of the load is transmitted to the soil on the sides of the hole. To secure the best results, the sand should be fine, sharp, clean, and of uniform size.

289. When the piles are driven primarily to compact the soil, it is customary to load them and also the soil between them, either by cutting the piles off near the surface and laying a tight platform of timber on top of them (see § 320), or by depositing a bed of concrete between and over the heads of the piles (see § 319).

If the soil is very soft or composed largely of sand, this method is ineffective; in which case long piles are driven as close together as is necessary, the supporting power being derived either from the resting of the piles upon a harder substratum or from the buoyancy due to immersion in the semi-liquid soil. This method of securing a foundation by driving long piles is very expensive, and is seldom resorted to for buildings, since it is generally more economical to increase the area of the foundation.

290. Layers of Sand. If the soil is very soft, it may be excavated and replaced by sand. The method of using sand for piles has been described in § 288, which see. The opportunities for the use of sand in foundations are numerous, and the employment of it would, in many constructions, promote economy and stability. The simplest method of using sand for this purpose is to excavate a trench or pit to the proper depth, and fill it by depositing successive layers of sand, each of which should be thoroughly settled by a heavy beetle before laying the next. To cause the sand to pack firmly, it should be slightly moistened before being placed in the trench.

Sand, when used in this way, possesses the valuable property of assuming a new position of equilibrium and stability should the soil on which it is laid yield at any of its points; not only does this take place along the base of the sand bed, but also along its edges or sides. The bed of sand must be thick enough to distribute the pressure on its upper surface over the entire base. There is no way of telling what this thickness should be, except by trial.

291. The following examples, cited by Trautwine,* are interesting as showing the surprising effect of even a thin layer of sand or gravel:

"Some portions of the circular brick aqueduct for supplying Boston with water gave a great deal of trouble when its trenches passed through running quicksands and other treacherous soils. Concrete was tried, but the wet quicksand mixed itself with it and *killed* it. Wooden cradles, etc., also failed; and the difficulty was overcome by simply depositing in the trenches about two feet in depth of strong gravel.

"Smeaton mentions a stone bridge built upon a natural bed of gravel only about 2 feet thick, overlying deep mud so soft that an iron bar 40 feet long sank to the head by its own weight. Although a wretched precedent for bridge building, this example illustrates the bearing power of a thick layer of well-compacted gravel."

ART. 2. DESIGNING THE FOOTING.

292. LOAD TO BE SUPPORTED. The first step is to ascertain the load to be supported by the foundation. This load consists of three parts: (1) the building itself, (2) the movable loads on the floors and the snow on the roof, and (3) the part of the load that may be transferred from one part of the foundation to the other by the force of the wind.

293. The weight of the building is easily ascertained by calculating the cubical contents of all the various materials in the structure. If the weight is not equally distributed, care must be taken to ascertain the proportion to be carried by each part of the foundation. For example, if one vertical section of the wall is to contain a number of large windows while another will consist entirely of solid masonry, it is evident that the pressure on the foundation under the first section will be less than that under the second.

In this connection it must be borne in mind that concentrated pressures are not transmitted, undiminished, through a solid mass in the line of application, but spread out in successively radiating lines; hence, if any considerable distance intervenes between the foundation and the point of application of this concentrated load,

* Engineer's Pocket-book (ed. 1885), p. 684.

the pressure will be nearly or quite uniformly distributed over the entire area of the base. The exact distribution of the pressure can not be computed.

The following data will be useful in determining the weight of the structure :

TABLE 24.

WEIGHT OF MASONRY.

KIND OF MASONRY.	WEIGHT IN LBS. PER CU. FT.
Brick-work, pressed brick, thin joints	145
" ordinary quality	125
" soft brick, thick joints	100
Concrete, best	160
" porous	130
Granite or Limestone, well dressed throughout	165
" " " rubble, well dressed, with mortar	155
" " " " roughly dressed, with mortar	150
" " " " well dressed, dry	140
" " " " roughly dressed, dry	125
Mortar, dried	100
Sandstone. (Deduct $\frac{1}{10}$ from the result for the corresponding granite or limestone masonry as above.)	—

Ordinary lathing and plastering weighs about 10 lbs. per sq. ft. The weight of floors is approximately 10 lbs. per sq. ft. for dwellings; 25 lbs. per sq. ft. for public buildings; and 40 or 50 lbs. per sq. ft. for warehouses. The weight of the roof varies with the kind of covering, the span, etc. A shingle roof may be taken at 10 lbs. per sq. ft., and a roof covered with slate or corrugated iron at 25 lbs. per sq. ft.

294. The movable load on the floor depends upon the nature of the building. For dwellings, it does not exceed 10 lbs. per sq. ft.; for large office buildings, it is usually taken at 20 lbs. per sq. ft.; for churches, theatres, etc., the maximum load—a crowd of people—may reach 100 lbs. per sq. ft.; for stores, warehouses, factories,

etc., the load will be from 100 to 400 lbs. per sq. ft., according to the purposes for which they are used. Floors of large buildings in which dense crowds of people are liable to congregate may be loaded to 100 or even 150 lbs. per sq. ft. However, there is no probability that any such aggregation of load will come upon the foundations as would be represented by 150 pounds for every square foot of flooring in a building. The amount of moving load to be assigned for in any particular case is a matter of judgment.

The weight of the snow on the roof will vary from 0, for a building in a warm climate, to 20 lbs. per sq. ft., for one with a flat roof situated in the latitude of New England or northern Michigan.

Attention must be given to the manner in which the weight of the roof and floors is transferred to the walls. For example, if the floor joists of a warehouse run from back to front, it is evident that the back and front walls alone will carry the weight of the floors and of the goods placed upon them, and this will make the pressure upon the foundation under them considerably greater than under the other walls. Again, if a stone-front is to be carried on an arch or on a girder having its bearings on piers at each side of the building, it is manifest that the weight of the whole superincumbent structure, instead of being distributed equally on the foundation under the front, will be concentrated on that part of the foundation immediately under the piers.

295. The pressure of the wind against towers, tall chimneys, etc., will cause a concentration of the weight of the structure upon one side of the foundation. The maximum horizontal pressure of the wind is usually taken as 50 lbs. per sq. ft. on a flat surface perpendicular to the wind, and on a cylinder at about 30 lbs. per sq. ft. of the projection of the surface. The pressure upon an inclined surface, as a roof, is about 1 lb. per sq. ft. per degree of inclination to the horizontal. For example, if the roof has an inclination of 30° with the horizontal, the pressure of the wind will be about 30 lbs. per sq. ft.

The effect of the wind will be considered in §§ 301–4.

296. Area Required. Having determined the pressure which may safely be brought upon the soil, and having ascertained the weight of each part of the structure, the area required for the foundation is easily determined by dividing the latter by the former.

Then, having found the area of foundation, the base of the structure must be extended by footings of masonry, concrete, timber, etc., so as to (1) cover that area and (2) distribute the pressure uniformly over it. The two items will be considered in inverse order.

297. Center of Pressure and Center of Base. In constructing a foundation the object is not so much to secure an absolutely unyielding base as to secure one that will settle as little as possible, and uniformly. All soils will yield somewhat under the pressure of any building, and even masonry itself is compressed by the weight of the load above it. The pressure per square foot should, therefore, be the same for all parts of the building, and particularly of the foundation, so that the settlement may be uniform. This can be secured only when the axis of the load (a vertical line through the center of gravity of the weight) passes through the center of the area of the foundation. If the axis of pressure does not coincide exactly with the axis of the base, the ground will yield most on the side which is pressed most; and as the ground yields, the base assumes an inclined position, and carries the lower part of the structure with it, thus producing unsightly cracks, if nothing more.

The coincidence of the axis of pressure with the axis of resistance is of *first* importance. This principle is self-evident, and yet the neglect to observe it is the most frequent cause of failure in the foundations of buildings.

Fig. 50 is an example of the way in which this principle is violated. The shaded portion represents a heavily loaded exterior wall, and the light portion a lightly loaded interior wall. The foundations of the two walls are rigidly connected together at their intersection. The center of the load is under the shaded section, and the center of the area is under the interior of the wall; consequently the exterior wall is caused to incline outward, producing cracks at or near the corners of the building. Doubtless the two foundations are connected in the belief that an increase of the bearing surface is of first importance; but the true principle is that the coincidence of the axis of pressure with the axis of resistance is the most important.

Fig. 50.

Fig. 51 is another illustration of the same principle. The foundation is continuous under the opening, and hence the center of the foundation is to the left of the center of pressure; consequently the wall inclines to the right, producing cracks, usually over the opening.*

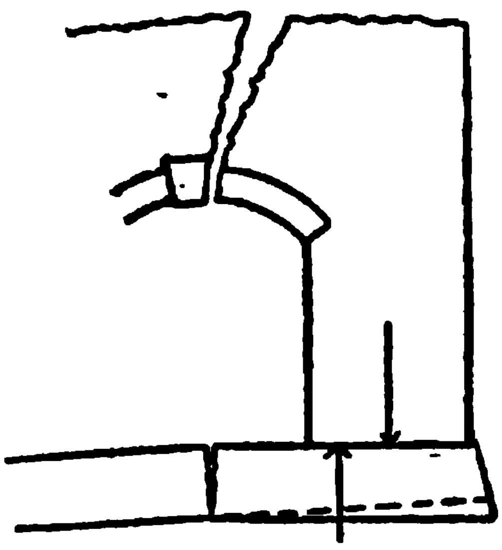


FIG. 51.

298. The center of the load can be made to fall inside of the center of foundation by extending the footings outwards, or by curtailing the foundations on the inside. The latter finds exemplification in the properly constructed foundation of a wall containing a number of openings. For example, in Fig. 52, if the foundation is uniform under the entire front, the center of pressure must be outside of the center of the base; and consequently the two side walls will incline outward, and show cracks over the openings. If the width of the foundation under the openings be decreased, or if this part of the foundation be omitted entirely, the center of pressure will fall inside of the center of base and the walls will tend to incline inwards, and hence be stable.

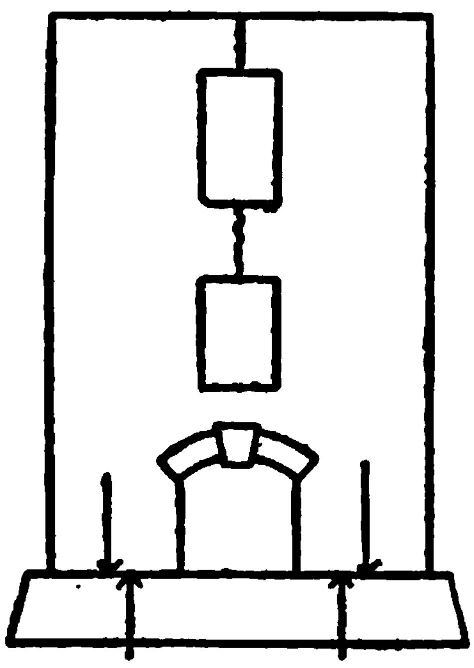


FIG. 52.

299. Conclusions. One conclusion to be drawn from the above examples is that the foundation of a wall should never be connected with that of another wall either much heavier or much lighter than itself. Both are equally objectionable.

A second conclusion is that the axis of the load should strike a little *inside* of the center of the area of the base, to make sure that it will not be *outside*. Any inward inclination of the wall is rendered impossible by the interior walls of the building, the floor-beams, etc.; while an outward inclination can be counteracted only by anchors and the bond of the masonry. A slight deviation of the axis of the load outward from the center of the base has a marked effect, and is not easily counteracted by anchors.

* For an account showing the violation of this principle in the construction of the Cooper Institute Building, New York City, and the method used to remedy it, see *Sanitary Engineer*, vol. xii. pp. 465–68.

The above conclusions may be summarized in the following principle: *All foundations should be so constructed as to compress the ground slightly* CONCAVE *upwards, rather than* CONVEX *upwards.* On even slightly compressible soils, a small difference in the pressure on the foundation will be sufficient to cause the bed to become convex upwards. At Chicago, an omission of 1 to 2 per cent. of the weight (by leaving openings) usually causes sufficient convexity to produce unsightly cracks. With very slight differences of pressure on the foundation, it is sufficient to tie the building together by careful bonding, by hoop-iron built in over openings, and by heavy bars built in where one wall joins another.

300. INDEPENDENT PIERS. The art of constructing foundations on compressible soil has been brought to a high degree of development by the architects of Chicago. The special feature of the practice in that city is what is called "the method of independent piers;" that is, each tier of columns, each pier, each wall, etc., has its own independent foundation, the area of which is proportioned to the load on that part.* The interior walls are fastened to the exterior ones by anchors which slide in slots. For a detailed account of the methods employed in one of the best and largest buildings erected there, see *Sanitary Engineer*, Dec. 10, 1885.

301. EFFECT OF THE WIND. Overturning. The preceding discussion refers to the total weight that is to come upon the foundation. The pressure of the wind against towers, tall chimneys, etc., transfers the point of application of the load to one side of the foundation. The method of computing the position of the center of the pressure on the foundation under the action of the wind is illustrated in Fig. 53, in which

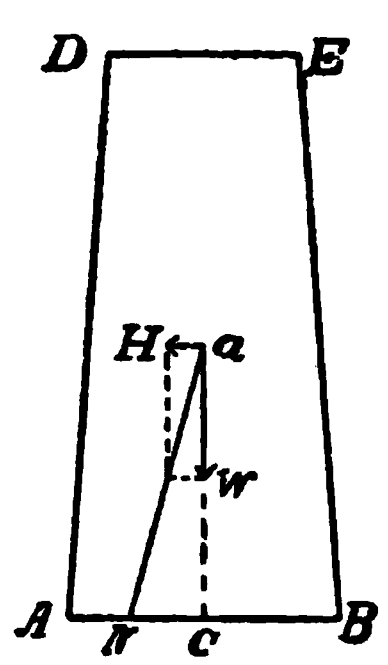


FIG. 53.

$A\ B\ E\ D$ represents a vertical section of the tower;

a is a point horizontally opposite the center of the surface exposed to the pressure of the wind and vertically above the center of gravity of the tower;

* This method was first made known to the public by Frederick Bauman, of Chicago, in a pamphlet entitled "The Method of Constructing Foundations on Isolated Piers," published by him in 1872. The above examples and principles are from that pamphlet.

C is the position of the center of pressure when there is no wind;
N is the center when the wind is acting.

For convenience, let

P = the maximum pressure on the foundation, per unit of area;
p = the pressure of the wind per unit of area (see § 295);
H = the total pressure of the wind against the exposed surface;
W = the weight of that part of the structure above the section considered,—in this case, $A\ B$;
S = the area of the horizontal cross section;
I = the moment of inertia of this section;
l = the distance $A\ B$;
h = the distance $a\ C$;
d = the distance $N\ C$;
M = the moment of the wind.

When there is no horizontal force acting, the load on $A\ B$ is uniform; but when there is a horizontal force acting—as, for example, the wind blowing from the right,—the pressure is greatest near A and decreases towards B. To find the law of the variation of this pressure, consider the tower as a cantilever beam. The maximum pressure at A will be that due to the weight of the tower *plus* the compression due to flexure; and the pressure at B will be the compression due to the weight *minus* the tension due to flexure. The uniform pressure due to the weight is $\frac{W}{S}$. The strain at A due to flexure is, by the principles of the resistance of materials, $\frac{Ml}{2I}$. Then the maximum pressure per unit of area at A is

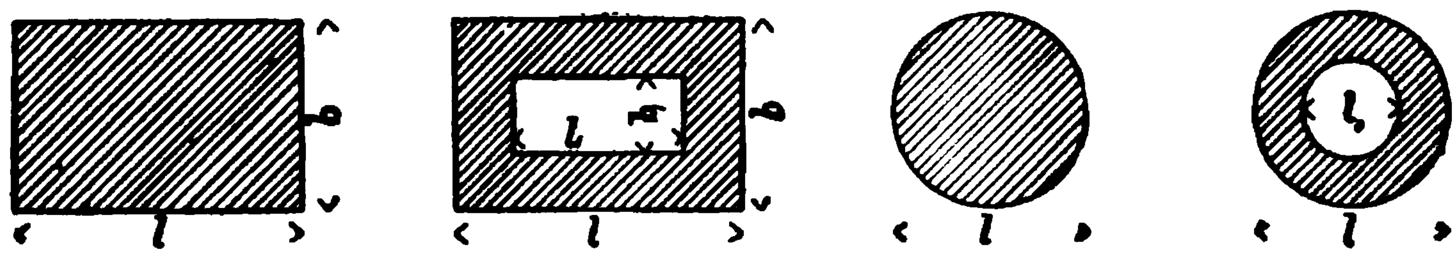
$$P = \frac{W}{S} + \frac{Ml}{2I}, \qquad (1)$$

and the minimum pressure at B is

$$P = \frac{W}{S} - \frac{Ml}{2I}. \qquad (2)$$

Equations (1) and (2) are perfectly general; they are applicable to any cross section, and also to any system of horizontal and vertical forces. In succeeding chapters they will be employed in finding the unit pressure in masonry dams, bridge piers, arches, etc.

The value of I in the above formulas is given in Fig. 54 for the sections occurring most frequently in practice. Notice that l is the dimension parallel to the direction of the wind, and b the dimension perpendicular to the direction of the wind.



$I = \frac{1}{12} b l^3$ $\quad I = \frac{1}{12}(b l^3 - b_1 l_1^3)$ $\quad I = \frac{1}{64} \pi l^4$ $\quad I = \frac{1}{64} \pi (l^4 - l_1^4)$

FIG. 54.

302. If the area of the section $A B$, Fig. 53, is a rectangle, $S = l b$, and $I = \frac{1}{12} b l^3$. Substituting these values in equation (1) gives

$$P = \frac{W}{l b} + \frac{6M}{b l^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The moment of the wind, M, is equal to the product of its total pressure, H, and the distance, h, of the center of pressure above the horizontal section considered; or $M = H . h$. H is equal to the pressure per unit of area, p, *multiplied by* the area of the surface exposed to the pressure of the wind. Substituting the above value of M in equation (3) gives

$$P = \frac{W}{l b} + \frac{6 H . h}{b l^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

To still further simplify the above formula, notice that Fig. 53 gives the proportion

$$H : W :: N C : a C,$$

from which

$$H . a C = W . N C;$$

or, changing the nomenclature,

$$H h = W d.$$

Notice that the last relation can also be obtained directly by the principle of moments. Substituting the value of $H . h$, as above, in equation (4) gives

$$P = \frac{W}{l b} + \frac{6 W d}{b l^2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

which is a convenient form for practical application.

An examination of equation (5) shows that when $d = NC = \frac{1}{6}l$, the maximum pressure at A is twice the average. Notice also that under these conditions the pressure at B is zero. This is equivalent to what is known, in the theory of arches, as the principle of the middle third. It shows that as long as the center of pressure lies in the middle third, the maximum pressure is not more than twice the average pressure, and that there is no tension at B.

The above discussion of the distribution of the pressure on the foundation is amply sufficient for the case in hand; however, the subject is discussed more fully in the chapter on Stability of Masonry Dams (see Chapter XIII).

303. The average pressure per unit on AB has already been adjusted to the *safe* bearing power of the soil, and if the maximum pressure at A does not exceed the *ultimate* bearing power, the occasional maximum pressure due to the wind will do no harm; but if this maximum exceeds or is dangerously near the ultimate strength of the soil, the base must be widened.

304. Sliding. The pressure of the wind is a force tending to slide the foundation horizontally. This is resisted by the friction caused by the weight of the entire structure, and also by the earth around the base of the foundation, and hence there is no need, in this connection, of considering this manner of failure.

305. Designing the Footing. The term *footing* is usually understood as meaning the bottom course or courses of masonry which extend beyond the faces of the wall. It will be used here as applying to the material—whether masonry, timber, or iron—employed to increase the area of the base of the foundation. Whatever the character of the soil, footings should extend beyond the face of the wall (1) to add to the stability of the structure and lessen the danger of the work's being thrown out of plumb, and (2) to distribute the weight of the structure over a larger area and thus decrease the settlement due to the compression of the ground. To serve the first purpose, footings must be securely bonded to the body of the wall; and to produce the second effect, they must have sufficient strength to resist the transverse strain to which they are exposed. In ordinary buildings the distribution of the weight is more important than adding to the resistance to overturning, and hence only the former will be considered here.

The area of the foundation may be increased until the inherent

bearing power of the area covered is sufficient to support the load (1) by extending the bottom courses of masonry, or (2) by the use of one or more layers of timbers, railroad rails, or steel I-beams, or (3) by resting the structure upon inverted masonry arches.

306. Off-sets of Masonry Footings. The area of the foundation having been determined and its center having been located with reference to the axis of the load (§ 297), the next step is to determine how much narrower each footing course may be than the one next below it. The projecting part of the footing resists as a beam fixed at one end and loaded uniformly. The load is the pressure on the earth or on the course next below. The off-set of such a course depends upon the amount of the pressure, the transverse strength of the material, and the thickness of the course.

To deduce a formula for the relation between these quantities, let

P = the pressure, in tons per square foot, at the bottom of the footing course under consideration;

R = the modulus of rupture of the material, in pounds per square inch;

p = the greatest possible projection of the footing course, in inches;

t = the thickness of the footing course, in inches.

The part of the footing course that projects beyond the one above it, is a cantilever beam uniformly loaded. From the principles of the resistance of materials, we know that the upward pressure of the earth against the part that projects *multiplied by* one half of the length of the projection *is equal to* the continued product of one sixth of the modulus of rupture of the material, the breadth of the footing course, and the square of the thickness. Expressing this relation in the above nomenclature and reducing, we get the formula

$$p = t\sqrt{\frac{R}{41.6\,P}}, \quad \dots \dots \dots \quad (6)$$

or, with sufficient accuracy,

$$p = \tfrac{1}{7}\,t\sqrt{\frac{R}{P}}. \quad \dots \dots \dots \quad (7)$$

Hence the projection available with any given thickness, or the thickness required for any given projection, may easily be computed

by equation (7). Notice that with the off-set given by the above formula the stone would be on the point of breaking.

307. The margin to be allowed for safety will depend upon the care used in computing the loads, in selecting the materials for the footing courses, and in bedding and placing them. If *all* the loads have been allowed for at their probable maximum value, and if the material is to be reasonably uniform in quality and laid with care, then a comparatively small margin for safety is sufficient; but if *all* the loads have not been carefully computed, and if the job is to be done by an unknown contractor, and neither the material nor the work is to be carefully inspected, then a large margin is necessary. As a general rule, it is better to assume, for each particular case, a factor of safety in accordance with the attendant conditions of the problem than blindly to use the results deduced by the application of some arbitrarily assumed factor. The following table is given for the convenience of those who may wish to use 10 as a factor of safety.

TABLE 25.

SAFE OFF-SET FOR MASONRY FOOTING COURSES, IN TERMS OF THE THICKNESS OF THE COURSE, USING 10 AS A FACTOR OF SAFETY.

For limitations, see § 308.

KIND OF STONE.	R, IN LBS. PER SQ. IN.	OFF-SET FOR A PRESSURE, IN TONS PER SQ. FT., ON THE BOTTOM OF THE COURSE, OF		
		0.5	1.0	2.0
Blue-stone flagging (see page 12)	2,700	3.6	2.6	1.8
Granite (see page 12)	1,800	2.9	2.1	1.5
Limestone (see page 12)	1,500	2.7	1.9	1.3
Sandstone (see page 12)	1,200	2.6	1.8	1.3
Slate (see page 12)	5,400	5.0	3.6	2.5
Best Hard Brick (see pages 40, 41)	1,500	2.7	1.9	1.3
Hard Brick (see pages 40, 41)	800	1.9	1.4	0.8
Concrete (see page 108) {1 Portland, 2 sand, 3 pebbles} 10 days old	150	0.8	0.6	0.4
Concrete (see page 108) {1 Rosendale, 2 sand, 3 pebbles} 10 days old	80	0.6	0.4	0.3

To illustrate the method of using the preceding table, assume that it is desired to determine the off-set for a limestone footing course when the pressure on the bed of the foundation is 1 ton per square foot, using 10 as a factor of safety. In the table, opposite

limestone, in next to the last column, we find the quantity 1.9. This shows that under the conditions stated, the off-set may be 1.9 times the thickness of the course.

If it is desired to use any other factor of safety, it is only necessary to substitute for R, in the preceding formula, the desired fractional part of that quantity as given in the second column of the above table. For example, assume that it is necessary to use limestone in the foundation, and that it is required to draw in the footing courses as rapidly as possible. Assume also that the pressure, P, on the base of the foundation is 2 tons per square foot. If the limestone is of the best, and if it is laid with great care, it will be sufficient to use 4 as a factor of safety. Under these conditions, equation (7) as above gives

$$p = \tfrac{1}{6}\, t \sqrt{\frac{\frac{1}{4} R}{P}} = \tfrac{1}{6}\, t \sqrt{\frac{\frac{1}{4} \times 1500}{2}} = 2.3\, t.$$

That is, the projection may be 2.3 times the thickness of the course.

308. Strictly, the above method is correct only when the footing is composed wholly of stones whose thickness is equal to the thickness of the course, and which project less than half their length, and are also well bedded. The values in the table agree very well with the practice of the principal architects and engineers for hammer-dressed stones laid in good cement mortar.

The preceding results will be applicable to built footing courses only when the pressure above the course is less than the safe strength of the mortar (see § 136 and § 157). The proper projection for rubble masonry lies somewhere between the values given for stone and those given for concrete. If the rubble consists of large stones well bedded in good strong mortar, then the values for this class of masonry will be but little less than those given in the table. If the rubble consists of small irregular stones laid with Portland or Rosendale cement mortar, the projection should not much exceed that given for concrete. If the rubble is laid in lime mortar, the projection of the footing course should not be more than half that allowed when cement mortar is used.

Notice that drawing in the footing courses decreases the area under pressure, and consequently increases the pressure per unit of area; hence the successive projections should decrease from the bottom towards the top.

309. Timber Footing. In very soft earth it would be inexpedient to use masonry footings, since the foundation would be very deep or occupy the space usually devoted to the cellar. One method of overcoming this difficulty consists in constructing a timber grating, sometimes called a *grillage*, by setting a series of heavy timbers firmly into the soil, and laying another series transversely on top of these. The timbers may be fastened at their intersections by spikes or drift-bolts (§ 381) if there is any possibility of sliding, which is unlikely in the class of foundations here considered. The earth should be packed in between and around the several beams. A flooring of thick planks, often termed a *platform*, is laid on top of the grillage to receive the lowest course of masonry. In extreme cases, the timbers in one or more of the courses are laid close together. Timber should never be used except where it will be always wet.

The amount that a course of timber may project beyond the one next above it can be determined by equation (7), page 208. Making R in that equation equal to 1,000—the value ordinarily used,—and solving, we obtain the following results for the *safe* projection: If the pressure on the foundation is 0.5 ton per square foot, the safe projection is 7.5 times the thickness of the course; if the pressure is 1 ton per square foot, the safe projection is 5.3 times the thickness of the course; and if the pressure is 2 tons per square foot, the safe projection is 3.7 times the thickness of the course. The above values give a factor of safety of about 10. To use any other factor, insert in equation (7), above, the corresponding fractional part of the ultimate transverse strength of the particular timber to be used, and solve.

310. This method of increasing the area of the footing is much used at New Orleans. The Custom-house at that place is founded upon a plank flooring laid 7 feet below the street pavement. A timber grillage, consisting of logs 12 inches in diameter laid side by side, is laid upon the floor, over which similar logs are placed transversely, 2 or 3 feet apart in the clear. The spaces are filled with concrete, and an additional thickness of 1 foot of concrete is placed over the whole. The grillage covers the entire site of the building—300 feet square. The settlement has been very great, and not uniform. This is not the fault of the general method employed, but is

due to the failure to proportion the area of each part of the foundation to the load to be supported.

311. Steel-rail Footing. Very recently, steel, usually in the form of railroad rails or I-beams, has been used instead of timber in foundations. The rails or I-beams are placed side by side, and concrete is rammed in between them.

Steel is superior to timber for this purpose, in that the latter can be used only where it is always wet, while the former is not affected by variations of wetness and dryness. Ten years' experience in this use of steel at Chicago shows that after a short time the surface of the metal becomes encased in a coating which prevents further oxidation. The most important advantage, however, in this use of steel is that the off-set can be much greater with steel or iron than with wood or stone; and hence the foundations may be shallow, and still not occupy the cellar space.

The proper off-set can be computed by a formula similar to that of § 306. Making these computations, it is found that if the pressure is 0.5 of a ton per square foot the off-set may be about 30 times the height of the rail; and that if the pressure is 2 tons per foot the off-set may be 15 times the height of the rail. There is no probability that in practice off-sets of such extreme lengths will ever be required.

In the foundation of the Rookery Building, Chicago, the steel-rail grillage consisted of four courses of rails, with bases nearly in contact, filled in between with cement mortar, the longitudinal and transverse courses alternating. The off-sets were 3 feet, the pressure being 2 tons per square foot.

312. Inverted Arch. Inverted arches are frequently built under and between the bases of piers, as shown in Fig. 55. Employed in this way, the arch simply distributes the pressure over a greater area; but it is not well adapted to this use, for (1) it is nearly impossible to prevent the end piers of a series from being pushed outward by the thrust of the arch, and (2) it is generally impossible, with inverted arches, to make the areas of the different parts of the foundation proportional to the load to be supported (see §

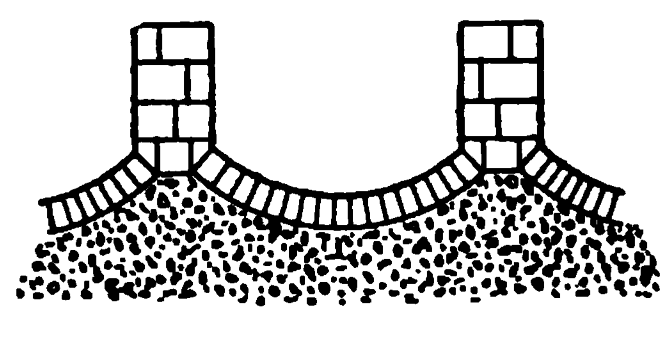


FIG. 55.

297). The only advantage the inverted arch has over masonry footings is in the shallower foundation obtained.

313. In a few cases masonry piers have been sunk to a solid substratum by excavating the material from the inside, and then resting arches on these piers. This is an expensive method, and has essentially the same objections as the inverted arch.

ART. 3. PREPARING THE BED.

314. ON ROCK. To prepare a rock bed to receive a foundation it is generally only necessary to cut away the loose and decayed portions of the rock, and to dress it to a plane surface as nearly perpendicular to the direction of the pressure as is practicable. If there are any fissures, they should be filled with concrete. A rock that is very much broken can be made amply secure for a foundation by the liberal use of good cement concrete. The piers of the Niagara Cantilever Bridge are founded upon the top of a bank of bowlders, which were first cemented together with concrete.

Sometimes it is necessary that certain parts of a structure start from a lower level than the others. In this case care should be taken (1) to keep the mortar-joints as thin as possible, (2) to lay the lower portions in cement, and (3) to proceed slowly with the work; otherwise the greater quantity of mortar in the wall on the lower portions of the slope will cause greater settling there and a consequent breaking of the joints at the stepping-places. The bonding over the off-sets should receive particular attention.

315. ON FIRM EARTH. For foundations in such earths as hard clay, clean dry gravel, or clean sharp sand, it is only necessary to dig a trench from 3 to 6 feet deep, so that the foundation may be below the disintegrating effect of frost. Provision should also be made for the drainage of the bed of the foundation.

With this class of foundations it often happens that one part of the structure starts from a lower level than another. When this is the case great care is required. All the precautions mentioned in the second paragraph of § 314 should be observed, and great care should also be taken so to proportion the load per unit of area that the settlement of the foundation may be uniform. This is difficult to do, since a variation of a few feet in depth often makes a great difference in the supporting power of the soil.

316. In Wet Ground. The difficulty in soils of this class is in disposing of the water, or in preventing the semi-liquid soil from running into the excavation. The difficulties are similar to those met with in constructing foundations under water—see Chapter XII. Three general methods of laying a foundation in this kind of soil will be briefly described.

317. Coffer-Dam. If the soil is only moderately wet—not saturated,—it is sufficient to inclose the area to be excavated with sheet piles (boards driven vertically into the ground in contact with each other). This curbing is a simple form of a coffer-dam (Art. 1, Chap. XII). The boards should be sharpened wholly from one side; this point being placed next to the last pile driven causes them to crowd together and make tighter joints. The sheeting may be driven by hand, by a heavy weight raised by a tackle and then dropped, or by an ordinary pile-driver (§§ 335–36). Unless the amount of water is quite small, it will be necessary to drive a double row of sheeting, breaking joints. It will not be possible to entirely prevent leaking. The water that leaks in may be bailed out, or pumped—either by hand or by steam (see § 395).

To prevent the sheeting from being forced inward, it may be braced by shores placed horizontally from side to side and abutting against wales (horizontal timbers which rest against the sheet piles). The bracing is put in successively from the top as the excavation proceeds; and as the masonry is built up, short braces between the sheeting and the masonry are substituted for the long braces which previously extended from side to side. Iron screws, somewhat similar to jack-screws, are used, instead of timber shores, in excavating trenches for the foundations of buildings, for sewers, etc.

If one length of sheeting will not reach deep enough, an additional section can be placed inside of the one already in position, when the excavation has reached a sufficient depth to require it. Ordinary planks 8 to 12 inches wide and 1½ or 2 inches thick are used.

For a more extended account of the use of coffer-dams, see Chapter XII—Foundations Under Water, Art. 1—Coffer-Dams.

318. In some cases the soil is more easily excavated if it is first drained. To do this, dig a hole—a sump—into which the water will drain and from which it may be pumped. If necessary, several sumps may be sunk, and deepened as the excavation proceeds.

Quicksand or soft alluvium may sometimes be pumped out along with the water by a centrifugal or a mud pump (§ 395 and § 448). On large jobs, such material is sometimes taken out with a clam-shell or Milroy dredge (§ 412).

319. Concrete. Concrete can frequently be used advantageously in foundations in wet soils. If the water can be removed, the concrete should be deposited in continuous layers, about 6 inches thick, and gently rammed until the water begins to ooze out on the upper surface (see § 153). If the water can not be removed, the concrete may be deposited under the water (see § 154), although it is more difficult to insure good results by this method than when the concrete is deposited in the open air.*

320. Grillage. If the semi-liquid soil extends to a considerable depth, or if the soft soil which overlies a solid substratum can not be removed readily, it is customary to drive piles at uniform distances over the area, and construct a grillage (see § 380) on top of them. This construction is very common for bridge abutments (Chapter XV). The piles should be sawed off (§ 378) below low-water, which usually necessitates a coffer-dam (§ 317, and Art. 1 of Chapter XII), and the excavation of the soil a little below the low-water line.

For a more extended account of this method of laying a foundation, see §§ 380–82.

321. In excavating shallow pits in sand containing a small amount of water, dynamite cartridges have been successfully used to drive the water out. A hole is bored with an ordinary auger and the cartridge inserted and exploded. The explosion drives the water back into the soil so far that, by working rapidly, the hole can be excavated and a layer of concrete placed before the water returns.

322. Conclusion. It is hardly worth while here to discuss this subject further. It is one on which general instruction can not be given. Each case must be dealt with according to the attendant circumstances, and a knowledge of the method best adapted to any given conditions comes only by experience.

* For the composition, cost, etc., of concrete, see Art. 2 of Chap. IV. pp. 102–12.

CHAPTER XI.

PILE FOUNDATIONS.

323. Definitions. Pile. Although a pile is generally understood to be a round timber driven into the soil to support a load, the term has a variety of applications which it will be well to explain.

Bearing Pile. One used to sustain a vertical load. This is the ordinary pile, and usually is the one referred to when the word *pile* is employed without qualification.

Sheet Piles. Thick boards or timbers driven in close contact to inclose a space, to prevent leakage, etc. Generally they are considerably wider than thick; but are sometimes square, in which case they are often called *close piles.*

False Pile. A timber added to a pile after driving, to supplement its length.

Foundation Pile. One driven to increase the supporting power of the soil under a foundation.

Screw Pile. An iron shaft to the bottom of which is attached a broad-bladed screw having only one or two turns.

Disk Pile. A bearing pile near the foot of which a disk is keyed or bolted to give additional bearing power.

Pneumatic Pile. A metal cylinder which is sunk by atmospheric pressure. This form of pile will be discussed in the next chapter (see § 431).

Art. 1. Descriptions, and Methods of Driving.

324. Iron Piles. Both cast and wrought iron are employed for ordinary bearing piles, sheet piles, and for cylinders. Iron cylinders are generally sunk either by dredging the soil from the inside (§ 415), or by the pneumatic process (see the next chapter, particularly §§ 431–35). For another method of employing iron cylinders, see §§ 384–85.

Cast-iron piles are beginning to be used as substitutes for common wooden ones. Lugs or flanges are usually cast on the sides of the piles, to which bracing may be attached for securing them in position. A wood block is laid upon the top of the pile to receive the hammer used in driving it; and, after being driven, a cap with a socket in its lower side is placed upon the pile to receive the load. The supporting power is sometimes increased by keying on an iron disk. The advantages claimed for cast-iron piles are: (1) they are not subject to decay; (2) they are more readily driven than wooden ones, especially in stony ground or stiff clay; and (3) they possess greater crushing strength, which, however, is an advantage only when the pile acts as a column (see § 355). The principal disadvantage is that they are deficient in transverse resistance to a suddenly applied force. This objection applies only to the handling of the piles before being driven, and to such as are liable, after being driven, to sudden lateral blows, as from floating ice, logs, etc.

Recently, rolled sections of wrought-iron have been employed to a limited degree for bearing-piles, but present prices prohibit an extended use of wrought-iron piles. It is possible that iron may take the place of wood for piles where they are alternately wet and dry, or where they are difficult to drive; but where the piles are always wet—as is usually the case in foundation work,—wood is as durable as iron; and hence, on account of cheapness, is likely to have the preference.

325. Screw Piles. These are generally wholly of iron, although the stem is sometimes wood. The screw pile usually consists of a rolled-iron shaft, 3 to 8 inches in diameter, having at its foot one or two turns of a cast-iron screw, the blades of which may vary from 1½ to 5 feet in diameter. The piles ordinarily employed for light-houses exposed to moderate seas or to heavy fields of ice have a shaft 3 to 5 inches in diameter and blades 3 to 4 feet in diameter, the screw weighing from 600 to 700 pounds. For bridge piers, the shafts are from 6 to 8 inches and the blades from 4 to 6 feet in diameter, the screw weighing from 1,500 to 4,000 pounds.

Screw piles were invented by Mitchell of Belfast, and are largely used in Europe, but not to any great extent in this country. They have been used in founding small light-houses on the sea-shore, for signal stations in marine surveying, for anchorage for buoys, and for various purposes inland.

For founding beacons, etc., the screw pile has the special advantage of not being drawn out by the upward force of the waves against the superstructure. Even when all cohesion of the ground is destroyed in screwing down a pile, a conical mass, with its apex at the bottom of the pile and its base at the surface, would have to be lifted to draw the pile out. The supporting power also is considerable owing to the increased bearing surface of the screw blade. Screw piles have, therefore, an advantage in soft soil. They could also be used advantageously in situations where the jar of driving ordinary piles might disturb the equilibrium of adjacent structures.

326. These piles are usually screwed into the soil by men working with capstan bars. Sometimes a rope is wound around the shaft and the two ends pulled in opposite directions by two capstans, and sometimes the screw is turned by attaching a large cog-wheel to the shaft by a friction-clutch, which is rotated by a worm-screw operated by a hand crank. Of course steam or horse-power could be used for this purpose.

The screw will penetrate most soils. It will pass through loose pebbles and stones without much difficulty, and push aside bowlders of moderate size. Ordinary clay does not present much obstruction; clean, dry sand gives the most difficulty. The danger of twisting off the shaft limits the depth to which they may be sunk. Screw piles with blades 4 feet in diameter have been screwed 40 feet into a mixture of clay and sand. The resistance to sinking increases very rapidly with the diameter of the screw; but under favorable circumstances an ordinary screw pile can be sunk very quickly. Screws 4 feet in diameter have, in less than two hours, been sunk by hand-labor 20 feet in sand and clay, the surface of which was 20 feet below the water. For depths of 15 to 20 feet, an average of 4 to 8 feet per day is good work for wholly hand-labor.

For an illustrated and detailed account of the founding of a railroad bridge pier on screw piles, see *Engineering News*, Vol. XIII. pp. 210–12.

327. **Disk Piles.** These differ but little from screw piles, a flat disk, instead of a screw, being keyed on at the foot of the iron stem. Disk piles are sunk by the water-jet (§ 343). One of the few cases in which they have been used in this country was in founding an ocean pier on Coney Island, near New York City. The shafts were wrought-iron, lap-welded tubes, 8⅝ inches outside diameter, in

sections 12 to 20 feet long; the disks were 2 feet in diameter and 9 inches thick, and were fastened to the shaft by set-screws. Many of the piles were 57 feet long, of which 17 feet was in the sand.*

328. Sand Piles. For an account of the method of using sand as piles, see § 288.

329. Sheet Piles. These are flat piles, which, being driven successively edge to edge, form a vertical or nearly vertical sheet for the purpose of preventing the materials of a foundation from spreading, or of guarding them against the undermining action of water. They may be made either of timber or iron. Ordinarily sheet piles are simply thick planks, sharpened and driven edge to edge. Sometimes they have a tongue on one edge and a corresponding groove on the other, to aid in guiding them into place while driving. When heavy timbers are employed as sheet piling, wooden blocks or iron lugs are fastened on the edges to assist in guiding them into position. Sheet piles should be sharpened wholly, or at least mainly, from one side, and the long edge placed next to the pile already driven. This causes them to crowd together and make comparatively close joints.

When a space is to be inclosed with sheet piling, two rows of guide piles are first driven at regular intervals of from 6 to 10 feet, and to opposite sides of these, near the top, are notched or bolted a pair of parallel string-pieces, or *wales*, from 5 to 10 inches square, so fastened to the guide piles as to leave a space between the wales equal to the thickness of the sheet piles. If the sheeting is to stand more than 8 or 10 feet above the ground, a second pair of wales is required near the level of the ground. The sheet piles are driven (§§ 334–45) between the wales, working from both ends towards the middle of the space between a pair of guide piles, so that the last or central pile acts as a wedge to tighten the whole.

330. Wooden Bearing Piles. Spruce and hemlock answer very well, in soft or medium soils, for foundation piles or for piles always under water; the hard pines, elm, and beech, for firmer soils; and the hard oaks, for still more compact soils. Where the pile is alternately wet and dry, white or post oak and yellow or southern pine are generally used.

* For a detailed and illustrated description of this work, see an article by Charles Macdonald, C.E., in Trans. Am. Soc. of C. E., Vol. VIII. pp. 227–87.

Piles should never be less than 8 inches in diameter at the small end and never more than 18 inches at the large end. Specifications usually require that these dimensions shall not be less than 10 nor more than 14 inches respectively. Piles should be straight-grained, should be trimmed close, and should have the bark removed.

331. Specifications for Piles. The ordinary specifications are about as follows :*

Piles, whether used in foundations, trestle-work, or pile bridges, shall be of good quality, sound, white oak, or such other timber as the engineer may direct, not less than ten inches (10") in diameter at the smaller end and 14 inches (14") at the larger, and of such lengths as the engineer may require. They must be straight-grained, must be trimmed close, and must have all the bark taken off before being driven. They must be cut off square at the butt, and be properly sharpened. If required by the engineer, the point shall be shod with iron shoes [see § 332], and the head hooped with iron bands of approved size and form [see § 332], which will be paid for by the pound.

332. Pile Caps and Shoes. To prevent bruising and splitting in driving, 2 or 3 inches of the head is usually chamfered off. As an additional means of preventing splitting, the head is often hooped with a strong iron band, 2 to 3 inches wide and ½ to 1 inch thick. The expense of removing these bands and of replacing the broken ones, and the consequent delays, led to the introduction, recently, of a cap for the protection of the head of the pile. The cap consists of a cast-iron block with a tapered recess above and below, the chamfered head of the pile fitting into the lower recess and a cushion piece of hard wood, upon which the hammer falls, fitting into the upper one. The cap preserves the head of the pile, adds to the effectiveness of the blows (§ 361), and keeps the pile head in place to receive the blows of the hammer.

A further advantage of the pile cap is that it saves piles. In hard driving, without the cap the head is crushed or broomed to such an extent that the pile is adzed or sawed off several times before it is completely driven, and often after it is driven a portion of the head must be sawed off to secure sound wood upon which to rest the grillage or platform (§ 380). In ordering piles for any special work where the driving is hard, allowance must be made for this loss.

Piles are generally sharpened before being driven, and some-

* See also "Piling" in the general specifications for railway masonry, as given in Appendix I.

times, particularly in stony ground, the point is protected by an iron shoe. The shoe may be only two V-shaped loops of bar iron placed over the point, in planes at right angles to each other, and spiked to the piles; or it may be a wrought or cast iron socket, of which there are a number of forms on the market.

333. Splicing Piles. It frequently happens, in driving piles in swampy places, for false-works, etc., that a single pile is not long enough, in which case two are spliced together. A common method of doing this is as follows:* after the first pile is driven its head is cut off square, a hole 2 inches in diameter and 12 inches deep is bored in its head, and an oak treenail, or dowel-pin, 23 inches long, is driven into the hole; another pile, similarly squared and bored, is placed upon the lower pile, and the driving continued. Spliced in this way the pile is deficient in lateral stiffness, and the upper section is liable to bounce off while driving. It is better to reinforce the splice by flatting the sides of the piles and nailing on, with say 8-inch spikes, four or more pieces 2 or 3 inches thick, 4 or 5 inches wide, and 4 to 6 feet long. In the erection of the bridge over the Hudson at Poughkeepsie, N. Y., two piles were thus spliced together to form a single one 130 feet long.

Piles may be made of any required length or cross-section by bolting and fishing together, sidewise and lengthwise, a number of squared timbers. Such piles are frequently used as guide piles in sinking pneumatic caissons (§ 436). Hollow-built piles, 40 inches in diameter and 80 feet long, were used for this purpose in constructing the St. Louis Bridge (§ 457). They were sunk by pumping the sand and water from the inside of them with a sand pump (§ 448).

334. PILE-DRIVING MACHINES. Pile-driving machines may be classified according to the character of the driving power, which may be (1) a falling weight, (2) the force of an explosive, or (3) the erosive action of a jet of water. Piles are sometimes set in holes bored with a well-auger, and the earth rammed around them. This is quite common in the construction of small highway bridges in the prairie States, a 10- or a 12-inch auger being generally used. The various pile-driving machines will now be briefly described and compared.

* See "Piling" in the General Specifications for Railroad Masonry, as given in Appendix I.

335. Drop-hammer Pile-driver. The usual method of driving piles is by a succession of blows given with a heavy block of wood or iron—called a ram, monkey, or hammer—which is carried by a rope or chain passing over a pulley fixed at the top of an upright frame, and allowed to fall freely on the head of the pile. The machine for doing this is called a drop-hammer pile-driver, or a monkey pile-driver—usually the former. The machine is generally placed upon a car or scow.

The frame consists of two uprights, called *leaders*, from 10 to 60 feet long, placed about 2 feet apart, which guide the falling weight in its descent. The leaders are either wooden beams or iron channel-beams, usually the former. The hammer is generally a mass of iron weighing from 500 to 4,000 pounds (usually about 2,000) with grooves in its sides to fit the guides and a staple in the top by which it is raised. The rope employed in raising the hammer is usually wound up by a steam-engine placed on the end of the scow or car, opposite the leaders.

A car pile-driver is made especially for railroad work, the leaders resting upon an auxiliary frame, by which piles may be driven 14 to 16 feet in advance of the end of the track; and the frame is pivoted so that piles may be driven on either side of the track. This method of pivoting the frame carrying the leaders is also sometimes applied to a machine used in driving piles for foundations.

In railroad construction, it is not possible to use the pile-driving car with its steam-engine in advance of the track; hence, in this kind of work, the leaders are often set on blocking and the hammer is raised by horses hitched directly to the end of the rope. Portable engines also are sometimes used for this purpose. Occasionally the weight is raised by men with a windlass, or by pulling directly on the rope.

A machine used for driving sheet piles differs from that described above in one particular, viz.: it has but one leader, in front of which the hammer moves up and down. With this construction, the machine can be brought close up to the wall of a coffer-dam (§ 317 and § 390), and the pile already driven does not interfere with the driving of the next one.

336. There are two methods of detaching the weight, *i. e.*, of letting the hammer fall: (1) by a nipper, and (2) by a friction-clutch.

1. The *nipper* consists of a block which slides freely between the leaders and which carries a pair of hooks, or tongs, projecting from its lower side. The tongs are so arranged that when lowered on to the top of the hammer they automatically catch in the staple in the top of the hammer, and hold it while it is being lifted, until they are disengaged by the upper ends of the arms striking a pair of inclined surfaces in another block, the *trip*, which may be placed between the leaders at any elevation, according to the height of fall desired.

With this form of machine, the method of operation is as follows: The pile being in place, with the hammer resting on the head of it and the tongs being hooked into the staple in the top of the hammer, the rope is wound up until the upper ends of the tongs strike the trip, which disengages the tongs and lets the hammer fall. As the hoisting rope is unwound the nipper block follows the hammer, and, on reaching it, the tongs automatically catch in the staple, and the preceding operations may be repeated. This method is objectionable owing to the length of time required (*a*) for the nipper to descend after the hammer has been dropped, and (*b*) to move the trip when the height of fall is changed. Some manufacturers of pile-driving machinery remove the last objection by making an adjustable trip which is raised and lowered by a light line passing over the top of the leaders. This is a valuable improvement.

When the rope is wound up by steam, the maximum speed is from 6 to 14 blows per minute, depending upon the distance the hammer falls. The speed can not be increased by the skill of the operator, although it could be by making the nipper block heavier.

2. The method by using a *friction-clutch*, or friction-drum, as it is often called, consists in attaching the rope permanently to the staple in the top of the hammer, and dropping the hammer by setting free the winding drum by the use of a friction-clutch. The advantages of this method are (*a*) that the hammer can be dropped from any height, thus securing a light or heavy blow at pleasure; and (*b*) that no time is lost in waiting for the nipper to descend and in adjusting the trip.

When the rope is wound up by steam, the speed is from 20 to 30 blows per minute, but is largely dependent upon the skill of the man who controls the friction-clutch. The hammer is caught on the rebound, is elevated with the speed of a falling body, and hence

the absolute maximum speed is attained. The rope, by which the hammer is elevated, retards the falling weight; and hence, for an equal effect, this form requires a heavier hammer than when the nipper is used. Although the friction-drum pile-driver is much more efficient, it is not as generally used as the nipper driver. The former is a little more expensive in first cost.

337. Steam-hammer Pile-driver. As regards frequency of use, the next machine is probably the steam-hammer pile-driver, invented by Nasmyth* in 1839. It consists essentially of a steam cylinder (stroke about 3 feet), the piston-rod of which carries a weight of about 3,500 pounds. The steam-cylinder is fastened to and between the tops of two I-beams about 8 to 10 feet long, the beams being united at the bottom by a piece of iron in the shape of a frustum of a cone, which has a hole through it. The under side of this connecting piece is cut out so as to fit the top of the pile. The striking weight, which works up and down between the two I-beams as guides, has a cylindrical projection on the bottom which passes through the hole in the piece connecting the feet of the guides and strikes the pile. The steam to operate the hammer is conveyed from the boiler through a flexible tube. Fig. 56 shows the striking weight of the latest form of steam-hammer. It differs from that described above in having four rods for guides, instead of the two I-beams.

The whole mechanism can be raised and lowered by a rope passing over a pulley in the top of the leaders. After a pile has been placed in position for driving, the machine is lowered upon the top of it and entirely let go, the pile being its only support. When steam is admitted below the piston, it rises, carrying the striking weight with it, until it strikes a trip, which cuts off the steam, and the hammer falls by its own weight. At the end of the down stroke the valves are again

FIG. 56.

* It is ordinarily called Nasmyth's hammer, but Bourdon should at least share the credit (see *Engineering News*, vol. xiii. pp. 59, 60).

automatically reversed, and the stroke repeated. By altering the adjustment of this trip-piece, the length of stroke (and thus the force of the blows) can be increased or diminished. The admission and escape of steam to and from the cylinder can also be controlled directly by the attendant, and the number of blows per minute is increased or diminished by regulating the supply of steam. The machine can give 60 to 80 blows per minute.

338. Drop-hammer vs. Steam-hammer. The drop-hammer is capable of driving the pile against the greater resistance. The maximum fall of the drop-hammer is 40 or 50 feet, while that of the steam-hammer is about 3 feet. The drop-hammer ordinarily weighs about 1 ton, while the striking weight of the steam-hammer usually weighs about 1½ tons. The energy of the maximum blow of the drop-hammer is 45 foot-tons (= 45 ft. × 1 ton), and the energy of the maximum blow of the steam-hammer is 4.5 foot-tons (= 3 ft. × 1½ tons). The energy of the maximum blow of the drop-hammer is, therefore, about 10 times that of the steam-hammer.

However, the effectiveness of a blow does not depend alone upon its energy. A considerable part of the energy is invariably lost by the compression of the materials of the striking surfaces, and the greater the velocity the greater this loss. An extreme illustration of this would be trying to drive piles by shooting rifle-bullets at them. A 1-ton hammer falling 45 ft. has 10 times the energy of a 1½-ton hammer falling 3 ft., but in striking, a far larger part of the former than of the latter is lost by the compression of the pile head. In constructing the foundation of the Brooklyn dry dock, it was practically demonstrated that "there was little, if any, gain in having the fall more than 45 feet." The loss due to the compression depends upon the material of the pile, and whether the head of it is bruised or not. The drop-hammer, using the pile-cap and the friction-drum, can drive a pile against a considerably harder resistance than the steam-hammer.

It is frequently claimed that the steam-hammer can drive a pile against a greater resistance than the drop-hammer. As compared with the old style drop-hammer, *i. e.*, without the friction-drum and the pile-cap, this is probably true. The striking of the weight upon the head of the pile splits and brooms it very much, which materially diminishes the effectiveness of the blow. In hard driving

with the drop-hammer, without the pile-cap, the heads of the piles, even when hooped, will crush, bulge out, and frequently split for many feet below the hoop. For this reason, it is sometimes specified that piles shall not be driven with a drop-hammer.

The rapidity of the blows is an important item as affecting the efficiency of a pile-driver. If the blows are delivered rapidly, the soil does not have sufficient time to recompact itself about the pile. With the steam-driver the blows are delivered in such quick succession that it is probable that a second blow is delivered before the pile has recovered from the distortion produced by the first, which materially increases the effectiveness of the second blow. In this respect the steam-hammer is superior to the drop-hammer, and the friction-clutch driver is superior to the nipper driver.

In soft soils, the steam-hammer drives piles faster than either form of the drop-hammer, since after being placed in position on the head of the pile it pounds away without the loss of any time.

339. In a rough way the first cost of the two drivers—exclusive of scow or car, hoisting engine, and boiler, which are the same in each—is about $80 for the drop-hammer driver, and about $800 for the steam-driver. Of course these prices will vary greatly. The per cent. for wear and tear is greater for the drop-hammer than for the steam-hammer. For work at a distance from a machine-shop the steam-driver is more liable to cause delays, owing to breakage of some part which can not be readily repaired.

340. Gunpowder Pile-driver. This machine was invented by Shaw, of Philadelphia, in 1870. The expansive force of gunpowder is utilized both in driving the pile and in raising the ram. The essential parts of the machine are the *ram* and *gun*. The former consists of a mass of iron weighing generally about 1,500 pounds, which terminates below in a sort of piston; this piston fits tightly into a chamber in another mass of iron, the gun. The ram travels between vertical guides much as in the other machines; and the gun and ram are hoisted as is the steam-hammer. The ram having been raised to the top of the guides, and the gun placed upon the top of the pile, a cartridge of from 1 to 3 ounces of gunpowder is placed in the cylinder, or gun, and the ram is allowed to descend. The piston enters the cylinder, compresses the air, and generates heat enough to ignite the cartridge, when the expansive force of

the powder forces the pile down and the ram up. A cartridge is thrown into the gun each time as the ram ascends. The rapidity of the blows is limited by the skill of the operator and by the heating of the gun. Thirty to forty blows, of from 5 to 10 feet each, can be made per minute.

341. The only advantage of this machine is that the hammer does not come in contact with the head of the pile, and hence does not injure it. The disadvantages are (1) that it is of no assistance in handling the pile; (2) that it is not economical; (3) that the gases soon destroy the gun; (4) that a leakage of gas occurs as the gun gets hot, which renders it less efficient as the rapidity of firing is increased; and (5) that the gun gets so hot as to explode the cartridge before the descent of the ram, which, of course, is an entire loss of the explosive. Its first cost is great. It is not now used.

342. Driving Piles with Dynamite. It has been proposed to drive piles by exploding dynamite placed directly upon the top of the pile. It is not known that this method has been used except in a few instances. It would be a slow method, but might prove valuable where only a few piles were to be driven by saving the transportation of a machine; or it might be employed in locations where a machine could not be operated. The higher grades of dynamite are most suitable for this purpose.*

343. Driving Piles with Water Jet. Although the water jet is not strictly a pile-driving machine, the method of sinking piles by its use deserves careful attention, because it is often the cheapest and sometimes the only means by which piles can be sunk in mud, silt, or sand.

The method is very simple. A jet of water is forced into the soil just below the point of the pile, thus loosening the soil and allowing the pile to sink, either by its own weight or with very light blows. The water may be conveyed to the point of the pile through a flexible hose held in place by staples driven into the pile; and after the pile is sunk, the hose may be withdrawn for use again. An iron pipe may be substituted for the hose. It seems to make very little difference, either in the rapidity of the sinking or in the accuracy with which the pile preserves its position, whether the nozzle is exactly under the middle of the pile or not.

* For a brief description of explosives, see pp. 119-24.

The water jet seems to have been first used in engineering in 1852, at the suggestion of General Geo. B. McClellan. It has been extensively employed on the sandy shores of the Gulf and South Atlantic States, where the compactness of the sand makes it difficult to obtain suitable foundations for light-houses, wharves, etc. Another reason for its use in that section is, that the palmetto piles —the only ones that will resist the ravages of the teredo—are too soft to withstand the blows of the drop-hammer pile-driver. By employing the water jet the necessity for the use of the pile-hammer is removed, and consequently palmetto piles become available. The jet has also been employed in a great variety of ways to facilitate the passage of common piles, screw and disk piles, cylinders, caissons, etc., etc., through earthy material.*

344. The efficiency of the jet depends upon the increased fluidity given to the material into which the piles are sunk, the actual displacement of material being small. Hence the efficiency of the jet is greatest in clear sand, mud, or soft clay; in gravel, or in sand containing a large percentage of gravel, or in hard clay, the jet is almost useless. For these reasons the engine, pump, hose, and nozzle should be arranged to deliver large quantities of water with a moderate force, rather than smaller quantities with high initial velocity. In gravel, or in sand containing considerable gravel, some benefit might result from a velocity sufficient to displace the pebbles and drive them from the vicinity of the pile; but it is evident that any practicable velocity would be powerless in gravel, except for a very limited depth, or where circumstances favored the prompt removal of the pebbles.

The error most frequently made in the application of the water jet is in using pumps with insufficient capacity. Both direet-acting and centrifugal pumps are frequently employed. The former affords the greater power; but the latter has the advantage of a less first cost, and of not being damaged as greatly by sand in the water used.

The pumping plant used in sinking the disk-piles for the Coney Island pier (see § 327), "consisted of a Worthington pump with a 12-inch steam cylinder, 8½-inch stroke, and a water cylinder 7½ inches in diameter. The suction hose was 4 inches in diameter,

* See a pamphlet—"The Water Jet as an Aid to Engineering Construction"—published (1881) by the Engineer Department of the U. S. Army.

and the discharge hose, which was of four-ply gum, was 3 inches. The boiler was upright, 42 inches in diameter, 8 feet high, and contained 62 tubes 2 inches in diameter. An abundance of steam was supplied by the boiler, after the exhaust had been turned into the smoke-stack and soft coal used as fuel. An average of about 160 pounds of coal was consumed in sinking each pile. With the power above described, it was found that piles could be driven in clear sand at the rate of 3 feet per minute to a depth of 12 feet; after which the rate of progress gradually diminished, until at 18 feet a limit was reached beyond which it was not practicable to go without considerable loss of time. It frequently happened that the pile would 'bring up' on some tenacious material which was assumed to be clay, and through which the water jet, unaided, could not be made to force a passage. In such cases it was found that by raising the pile about 6 inches and allowing it to drop suddenly, with the jet still in operation, and repeating as rapidly as possible, the obstruction was finally overcome; although in some instances five or six hours were consumed in sinking as many feet." *

In the shore-protection work on the Great Lakes, under the direction of the United States Army engineers, the pumping plant "consisted of a vertical tubular boiler, with an attached engine having an 8×12-inch cylinder, and giving about 130 revolutions per minute to a 42-inch driving-wheel. A No. 4 Holly rotary pump, with 18-inch pulley, was attached by a belt to the driving-wheel of the engine, giving about 300 revolutions per minute to the pump. The pump was supplied with a 4-inch suction pipe, and discharged through a 3-inch hose about 50 feet in length. The hose was provided with a nozzle 3 feet in length and 2 inches in diameter. The boiler, engine, pump, and pile-driver were mounted on a platform 12 feet in width and 24 feet in length." †

345. Jet vs. Hammer. It is hardly possible to make a comparison between a water-jet and a hammer pile-driver, as the conditions most favorable for each are directly opposite. For example, sand yields easily to the jet, but offers great resistance to driving with the hammer; on the other hand, in stiff clay the hammer is much

* Chas. McDonald, in Trans. Am. Soc. of C. E., vol. viii. pp. 227–37.

† "The Water-Jet as an Aid to Engineering Construction," p. 11;—a pamphlet published (1881) by the Engineer Department of the U. S. Army.

more expeditious. For inland work the hammer is better, owing to the difficulty of obtaining the large quantities of water required for the jet; but for river and harbor work the jet is the most advantageous. Under equally favorable conditions there is little or no difference in cost or speed of the two methods.*

The jet and the hammer are often advantageously used together, especially in stiff clay. The efficiency of the water-jet can be greatly increased by bringing the weight of the pontoon upon which the machinery is placed, to bear upon the pile by means of a block and tackle.

346. Cost of Piles. At Chicago and at points on the Mississippi above St. Louis, *pine piles* cost from 10 to 15 cents per lineal foot, according to length and location. *Soft-wood piles,* including rock elm, can be had in almost any locality for 8 to 10 cents per foot. *Oak piles* 20 to 30 feet long cost from 10 to 12 cents per foot; 30 to 40 feet long, from 12 to 14 cents per foot; 40 to 60 feet long, from 20 to 30 cents per foot.

347. Cost of Pile Driving. There are many items that affect the cost of work, which can not be included in a brief summary, but which must not be forgotten in using such data in making estimates. Below are the details for the several classes of work.

348. Railroad Construction. The following table is a summary of the cost, to the contractor, of labor in driving piles (exclusive of hauling) in the construction of the Chicago branch of the Atchison, Topeka and Santa Fé R. R. The piles were driven, ahead of the track, with a horse-power drop-hammer weighing 2,200 pounds. The average depth driven was 13 feet. The table includes the cost of driving piles for abutments for Howe truss bridges and for the false work for the erection of the same. These two items add considerably to the average cost. The contractor received the same price for all classes of work. The work was as varied as such jobs usually are, piles being driven in all kinds of soil. Owing to the large amount of railroad work in progress in 1887, the cost of material and labor was about 10 per cent. higher than the average of the year before and after. Cost of labor on pile-driver: 1 foreman at $4 per day, 6 laborers at $2, 2 teams at $3.50; total cost of labor = $23 per day.

* Report of Chief of Engineers U. S. A., 1883, pp. 1264–72.

COST OF PILE DRIVING IN RAILROAD CONSTRUCTION.

Number of piles included in this report	4,409
" " lineal feet included in this report	109,568
Average length of the piles, in feet	24.8
Number of days employed in driving	494′
" " lineal feet driven per day	221.8
Cost of driving, per pile	$2.58
" " " " foot	10.4 cents.

349. Railroad Repairs. The following are the data of pile driving for repairs to bridges on the Indianapolis, Decatur and Springfield R. R. The work was done from December 21, 1885, to January 5, 1886. The piles varied from 12 to 32 feet in length, the average being a little over 21 feet. The average distance driven was about 10 feet. The hammer weighed 1,650 pounds; the last fall was 37 feet, and the corresponding penetration did not exceed 2 inches. The hammer was raised by a rope attached to the draw-bar of a locomotive—comparatively a very expensive way.

TABLE 26.

COST OF PILES FOR BRIDGE REPAIRS.

ITEMS OF EXPENSE.	TOTAL.	PER PILE.	PER FOOT.
Labor: Loading and unloading piles, 7⅛ days	$16.00	$0.08	0.4 cts.
Bridge gang, driving, 12 days	153.75	0.78	3.7
Engine crew, transportation and driving, 18 days	45.90	0.23	1.1
Train crew, " " " "	71.50	0.37	1.6
Supplies: Engine supplies	23.49	0.13	0.5
6 pile rings and 2 plates	13.29	0.06	0.3
Repairs	11.04	0.05	0.3
Total expense for driving	$334.97	$1.70	7.9 cts.
Material: 4,192 feet oak piles at 13½ cts	$565.92	$2.86	13.5 cts.
TOTAL COST	$900.89	$4.56	21.4 cts.

On the same road, 9 piles, each 20 feet long, were driven 9 feet, for bumping-posts, with a 1,650-pound hammer dropping 17 feet. The hammer was raised with an ordinary crab-winch and single line, with double crank worked by four men. The cost for labor was 8.3 cents per foot of pile, and the total expense was 21.8 cents per foot.

350. Bridge Construction. The following table gives the cost of labor in driving the piles for the Northern Pacific R. R. bridge over the Red River, at Grand Forks, Dakota, constructed in 1887. The soil was sand and clay. The penetration under a 2,250-pound hammer falling 30 feet was from 2 to 4 inches. The foreman received $5 per day, the stationary engineer $3.50, and laborers $2.

TABLE 27.

COST OF LABOR IN DRIVING PILES IN BRIDGE CONSTRUCTION.

KIND OF LABOR.	PILE BRIDGE ON LAND.	TEMPORARY BRIDGE.	DRAW FENDER AND ICE BREAKER.	PIVOT PIER.	RIVER PIER.
Preparation and repair of plant	$68.95	$63.65	$53.50	$37.00	$61.60
Driving	432.70	252.92	430.50	215.45	565.80
Sawing and straightening	78.75		47.50	179.80*	131.90†
Total cost	$580.40	$316.57	$531.50	$432.25	$759.30
Number of piles in the structure	224	102	104	121	167
Total number of feet remaining in the structure	7,238	3,710	7,023	4,639	7,316
Average length of piles " " " "	32.3		38.2	38.4	43.8
Average length of piles cut off	1.1		4.1	6.6	3.7
Cost per foot of pile remaining in the structure	8.0 cts.	8.5 cts.	7.6 cts.	9.3 cts.	10.4 cts.

Average cost for driving, per foot remaining in the structure = 8.8 cents.

* Sawed off under 8 feet of water.

† Including $70.25 for excavating and bailing in order to get at the sawing.

351. Foundation Piles. The contract price for the foundation piles—white oak—for the railroad bridge over the Missouri River, at Sibley, Mo., was 22 cents per foot for the piles and 28 cents per foot for driving and sawing off below water. They were 50 feet long, and were driven in sand and gravel, in a coffer-dam 16 feet deep, by a drop-hammer weighing 3,203 pounds, falling 36 feet. The hammer was raised by steam power.

352. In the construction of a railroad in southern Wisconsin during 1885–87, the contract price—the lowest competitive bid—for the piles, in place, under the piers of several large bridges averaged as in the following table. The piles were driven in a strong current and sawed off under water, hence the comparatively great expense.

TABLE 28.

CONTRACT PRICE OF FOUNDATION PILES.

MATERIAL OF PILE.	KIND OF DRIVING.	CONTRACT PRICE PER LINEAL FOOT.	
		For Part remaining in Structure.	For Pile Heads Sawed off.
Rock Elm	Ordinary	40 cents	15 cents
Pine	"	40 "	20 "
Oak	"	48 "	25 "
Oak	Hard	50 "	30 "

353. In 1887 the contract price for piles in the foundations of bridge piers in the river at Chicago was 35 cents per foot of pile left in the foundation. This price covered cost of timber (10 to 15 cents), driving, and cutting off 12 to 14 feet below the surface of the water,—about 17 feet being left in the foundation.

The cost of driving and sawing off may be estimated about as follows: (17 + 13) feet of pile at 13 cents per foot = $3.90; 17 feet of pile, left in the structure, at 35 cents per foot = $5.95. $5.95 − $3.90 = $2.05 = the cost per pile of driving and sawing off, which is equivalent to nearly 7 cents per foot of total length of pile. In this case the waste or loss in the pile heads cut off adds considerably to the cost of the piles remaining in the structure. In making estimates this allowance should never be overlooked.

354. Harbor and River Work. In the shore-protection work at Chicago, done in 1882 by the Illinois Central R. R., a crew of 9 men, at a daily expense, for labor, of $17.25, averaged 65 piles per 10 hours in water 7 feet deep, the piles being 24 feet long and being driven 14 feet into the sand. The cost for labor of handling, sharpening, and driving, was a little over 26 cents per pile, or 1.9 cents per foot of distance driven, or 1.1 cents per foot of pile.* Both steam-hammers and water-jets were used, but not together. Notice that this is very cheap, owing (1) to the use of the jet, (2) to little loss of time in moving the driver and getting the pile exactly in the predetermined place, (3) to the piles not being sawed off, and (4) to the skill gained by the workmen in a long job.

On the Mississippi River, under the direction of the U. S. Army engineers, the cost in 1882 for labor for handling, sharpening, and driving, was $3.11 per pile, or 20 cents per foot driven. The piles were 35 feet long, the depth of water 15.5 feet, and the depth driven 13.6 feet. The water-jet and drop-hammer were used together. The large cost was due, in part at least, to the current, which was from 3 to 6 miles per hour.†

ART. 2. BEARING POWER OF PILES.

355. Two cases must be distinguished; that of columnar piles or those whose lower end rests upon a hard stratum, and that of ordinary bearing piles or those whose supporting power is due to the

* Report of the Chief of Engineers, U. S. A., for 1883, pp. 1266–70.

† *Ibid.*, p. 1260.

friction of the earth on the sides of the pile. In the first case, the bearing power is limited by the strength of the pile considered as a column; and, since the earth prevents lateral deflection, at least to a considerable degree, the strength of such a pile will approximate closely to the crushing strength of the material. This class of piles needs no further consideration here.

356. Methods of Determining Supporting Power. There are two general methods of determining the supporting power of ordinary bearing piles: first, by considering the relation between the supporting power and the length and size of the pile, the weight of the hammer, height of fall, and the distance the pile was moved by the last blow; or, second, by applying a load or direct pressure to each of a number of piles, observing the amount each will support, and expressing the result in terms of the depth driven, size of pile, and kind of soil. The first method is applicable only to piles driven by the impact of a hammer; the second is applicable to any pile, no matter how driven.

1. If the relation between the supporting power and the length and size of pile, the weight of the hammer, the height of fall, and the distance the pile was moved by the last blow can be stated in a formula, the supporting power of a pile can be found by inserting these quantities in the formula and solving it. The relation between these quantities must be determined from a consideration of the theoretical conditions involved, and hence such a formula is a *rational formula.*

2. By applying the second method to piles under all the conditions likely to occur in practice, and noting the load supported, the kind of soil, amount of surface of pile in contact with the soil, etc., etc., data could be collected by which to determine the supporting power of any pile. A formula expressing the supporting power in terms of these quantities is an *empirical formula.*

357. Rational Formulas. The deduction of a rational formula for the supporting power of a pile is not, strictly, an appropriate subject for mathematical investigation, as the conditions can not be expressed with mathematical precision. However, as there is already a great diversity of formulas in common use, which give widely divergent results, a careful investigation of the subject is necessary.

The present practice in determining the bearing power of piles is

neither scientific nor creditable. Many engineers, instead of inquiring into the relative merits of the different formulas, take an average of all the formulas they can find, and feel that they have a result based on the combined wisdom of the profession. This practice is exactly like that of the ship's surgeon who poured all his medicines into a black jug, and whenever a sailor was ailing gave him a spoonful of the mixture. Other engineers, knowing the great diversity and general unreliability of the formulas, reject them all and trust to their own experience and judgment. The self-reliant engineer usually chooses the latter course, while the timid one trusts to the former.

To correctly discriminate between the several formulas, it is necessary to have a clear understanding of all the conditions involved. The object of the following discussion is to discover the general principles which govern the problem.

353. When the ram strikes the head of the pile, the first effect is to compress both the head of the pile and the ram. The more the ram and pile are compressed the greater the force required, until finally the force of compression is sufficient to drive the pile through the soil. The amount of the pressure on the head of the pile when it begins to move, is what we wish to determine.

To produce a formula for the pressure exerted upon the pile by the impact of a descending weight, let

W = the weight of the ram, in tons;
w = " " " pile "
S = the section of the ram, in sq. ft.;
s = " " " pile " "
L = the length of the ram, in feet;
l = " " " pile "
E = the co-efficient of elasticity of the ram, in tons per sq. ft.;
e = " " " " " pile " " " "
h = the height of fall, in feet;
d = the penetration of the pile, *i. e.*, the distance the pile is moved by the last blow, in feet. The distance d is the amount the pile as a whole moves, and not the amount the top of the head moves. This can be found accurately enough by measuring the movement of a point, say, 2 or 3 feet below the head.
P = the pressure, in tons, which will just move the pile the very

small distance d,—that is to say, the pressure produced by the last blow; or, briefly, P may be called the supporting power.

Then Wh is the accumulated energy of the ram at the instant it strikes the head of the pile. This energy is spent (1) in compressing the ram, (2) in compressing the head of the pile, (3) in moving the pile as a whole against the resistance of the soil, (4) in overcoming the inertia of the pile, (5) in overcoming the inertia of the soil at the lower end of the pile, and (6) by the friction of the ram against guides and air. These will be considered in order.

1. The energy consumed in compressing the hammer is represented by the product of the mean pressure and the compression, or shortening, of the ram. The pressure at any point in a striking weight varies as the amount of material above that point; that is to say, the pressure at any point of the hammer varies inversely as its distance from the lower surface. The pressure at the lower surface is P, and that at the upper one is zero; hence the mean pressure is $\frac{1}{2}P$. From the principles of the resistance of materials, the compression, or the shortening, is $\frac{pL}{SE}$, in which p is the uniform pressure. From the above, $p = \frac{1}{2}P$. Consequently the shortening is $\frac{1}{2}\frac{PL}{SE}$.

If the fibers of the face of the ram are not seriously crushed, the mean pressure will be one half of the maximum pressure due to impact; or the mean pressure during the time the ram and pile are being compressed is $\frac{1}{2}P$. Then the energy consumed is $\frac{1}{4}\frac{P^2L}{SE}$. The yielding of the material of the ram is probably small, and might be omitted, but as it adds no complication, as will presently appear, it is included.

2. The mean pressure on the head of the pile is $\frac{1}{2}P$, as above. For simplicity assume that the pile is of uniform section throughout. To determine the shortening, notice that for the part of the pile above the ground the maximum pressure is uniform throughout, but that for the part under the surface the maximum pressure varies as some function of the length. If the soil were homogeneous, the pressure would vary about as the length in the ground; and

hence the shortening would be $\frac{1}{2}\frac{Pl}{se}$. But, remembering that the resistance is generally greater at the lower end than at the upper, and that any swaying or vibration of the upper end will still further diminish the resistance near the top, it is probable that the mean pressure is below the center. It will here be assumed that the mean pressure on the fibers of the pile is two thirds of that on the head, which is equivalent to assuming that the shortening is $\frac{2}{3}\frac{Pl}{se}$, when the pile is wholly immersed. If only a part of the pile is in contact with the soil, the shortening will be $\frac{Pl'}{se} + \frac{2}{3}\frac{Pl_1}{se} = \frac{P}{se}\left(l' + \frac{2}{3}l_1\right)$, in which l' is the exposed portion and l_1 the part immersed. For simplicity in the following discussion the shortening of the pile will be taken at $\frac{2}{3}\frac{Pl}{se}$. If a formula is desired for the case when the top projects above the ground, it will only be necessary to substitute $(\frac{3}{2}l' + l_1)$ for l in equations (1) and (2) below.

Then the energy lost in the compression of the pile is $\frac{1}{3}\frac{P^2 l}{se}$.

3. The energy represented by the penetration of the pile is $P d$.

4. In the early stage of the contact between the ram and the pile, part of the energy of the ram is being used up in overcoming the inertia of the pile; but in the last stage of the compression, this energy is given out by the stoppage of the pile. At most, the effect of the inertia of the pile is small; and hence it will be neglected.

5. The energy lost in overcoming the inertia of the soil at the lower end of the pile will vary with the stiffness of the soil and with the velocity of penetration. It is impossible to determine the amount of this resistance, and hence it can not be included in a formula. Omitting this element causes the formula to give too great a supporting power. The error involved can not be very great, and is to be covered by the factor of safety adopted.

6. The friction of the ram against the guides and against the air diminishes the effect of the blow, but the amount of this can not be computed. Omitting this element will cause the formula for the supporting power to give too great a result. The friction against the air increases very rapidly with the height of fall, and hence the

smaller the fall the more nearly will the formula give the true supporting power.

359. Equating the energy of the falling weight with that consumed in compressing the pile and ram, and in the penetration of the pile, as discussed in paragraphs 1, 2, and 3 above, we have

$$W h = \frac{1}{4}\frac{P^2 l}{S E} + \frac{1}{3}\frac{P^2 l}{s e} + P d. \quad . \quad . \quad . \quad . \quad (1)$$

Solving equation (1) gives

$$P = \sqrt{W h \frac{12\, S E s e}{3\, L s e + 4\, l S E} + \frac{36\, d^2 S^2 E^2 s^2 e^2}{(3\, L s e + 4\, l S E)^2}} - \frac{6\, d S E s e}{3\, L s e + 4\, l S E}. \quad . \quad (2)$$

An examination of equation (2) shows that the pressure upon the pile varies with the height of fall, the weight, section, length, and co-efficient of elasticity of both ram and pile, and with the penetration. It is easy to see that the weight of the ram and the height of the fall should be included. The penetration is the only element which varies with the nature of the soil, and so of course it also should be included. It is not so easy to see that the length, section, and co-efficient of elasticity of the material of the pile and ram should be included. If any one will try to drive a large nail into hard wood with a piece of leather or rubber intervening between the hammer and the head of the nail, he will be impressed with the fact that the yielding of the leather or rubber appreciably diminishes the effectiveness of the blow. Essentially the same thing occurs in trying to drive a large nail with a small hammer, except that in this case it is the yielding of the material of the hammer which diminishes the effect of the blow. In driving piles, the materials of the pile and ram act as the rubber in the first illustration; and, reasoning by analogy, those elements which determine the yielding of the materials of the pile and ram should be included in the formula. Obviously, then, the pressure due to impact will be greater the harder the material of the pile. Notice also that if the head of the pile is bruised, or "broomed," the yielding will be increased; and, consequently, the pressure due to the blow will be decreased.

360. The Author's Formula for Practice. To simplify equation (2), put

$$\frac{6\,S\,E\,s\,e}{3\,L\,s\,e+4\,l\,S\,E}=q,$$

and then equation (2) becomes

$$P=\sqrt{2\,q\,W\,h+q^2\,d^2}-q\,d. \quad . \quad . \quad . \quad . \quad (3)$$

Equation (3) can be simplified still further by computing q for the conditions as they ordinarily occur in practice. Of course, in this case it will only be possible to assume some average value for the various quantities. Assume the section of the pile to be 0.8 sq. ft.; the section of the ram, 2 sq. ft.; the length of the ram, 2.5 ft.; the length of the pile,* 25 ft.; the co-efficient of elasticity of the ram, 1,080,000 tons per sq. ft.; and the co-efficient of elasticity of the pile, 108,000 tons per sq. ft. (an average value for oak, elm, pine, etc., but not for palmetto and other soft woods). Computing the corresponding value of q, we find it to be 5,160; but to secure round numbers, we may take it at 5,000, which also gives a little additional security.

Equation (3) then becomes

$$P=100\left(\sqrt{W\,h+(50\,d)^2}-50\,d\right), \quad . \quad . \quad . \quad (4)$$

which is the form to be used in practice.

Equation (4) is approximate because of the assumptions made in deducing equation (1), and also because of the average value taken for q; but probably the error occasioned by these approximations is not material.

361. Notice that, since the co-efficient of elasticity of sound material was used in deducing the value of q, equation (4) is to be applied only on condition that the last blow is struck upon *sound* wood; and therefore the head of the test pile should be sawed off so as to present a solid surface for the last, or test, blow of the hammer. (*This limitation is exceedingly important.*) Since the penetration per blow can be obtained more accurately by taking the mean distance for two or three blows than by measuring the distance for a single one, it is permissible to take the mean penetration of two or

* The quantity to be used here is the length out of the ground *plus* about two thirds of the part in the ground (see paragraph 2 of § 353).

three blows; but their number and force should be such as not to crush the head of the pile.

In this connection the following table, given by Don. J. Whittemore, in the Transactions of the American Society of Civil Engineers, vol. xii. p. 442, to show the gain in efficiency of the driving power by cutting off the bruised or broomed head of the pile, is very instructive. The pile was of green Norway pine; the ram was of the Nasmyth type, and weighed 2,800 pounds.

TABLE SHOWING THE GAIN IN EFFICIENCY OF THE DRIVING POWER BY CUTTING OFF THE BROOMED HEAD OF THE PILE.

3d	ft. of	penetration	required	5	blows.
4th	"	"	"	15	"
5th	"	"	"	20	"
6th	"	"	"	29	"
7th	"	"	"	35	"
8th	"	"	"	46	"
9th	"		"	61	"
10th	"	"	"	73	"
11th	"	"	"	109	"
12th	"	"	"	153	"
13th	"	"	"	257	"
14th	"	"	"	684	"
Head of the pile adzed off.					
15th	ft. of	penetration	required	275	"
16th	"	"	"	572	"
17th	"	"	"	332	"
18th	"	"	"	825	"
Head of the pile adzed off.					
19th	ft. of	penetration	required	218	"
20th	"	"	"	275	"
21st	"	"	"	371	"
22d	"	"	"	378	"
Total number of blows,				5,228	

Notice that the average penetration per blow was 2½ times greater during the 15th foot than during the 14th; and nearly 4 times greater in the 19th than in the 18th. It does not seem unreasonable to believe that the first blows after adzing the head off were correspondingly more effective than the later ones; consequently, it is probable that the first blows for the 15th foot of penetration were more than 5 times as efficient as the last ones for the 14th foot, and also that the first blows for the 19th foot were 8 or 10 times more efficient than the last ones for the 18th foot. Notice also that since the head was only "adzed off," it is highly probable that the spongy wood was not entirely removed.

If the penetration for the last blow *before* the head was adzed off were used in the formula, the apparent supporting power would be very much greater than if the penetration for the first blow *after* adzing off is employed. This shows how unscientific it is to prescribe a limit for the penetration without specifying the accompanying condition of the head of the pile, as is ordinarily done.

362. Weisbach's Formula. Equation (2), page 238, is essentially equivalent to Weisbach's formula for the supporting power of a pile. Weisbach assumes that the pressure is uniform throughout, and obtains the formula*

$$P = \left(\frac{H H_1}{H + H_1}\right)\left(\sqrt{2\left(\frac{H + H_1}{H H_1}\right) h\, W + d^2} - d\right), \quad . \quad (5)$$

in which $H = \frac{S E}{L}$, and $H_1 = \frac{s e}{l}$.

363. Rankine's Formula. Equation (2), page 238, is also essentially equivalent to Rankine's formula; and differs from it, only because he assumes the pressure to vary directly as the length of the pile, and neglects the compression of the ram. Rankine's formula is †

$$P = \sqrt{\frac{4\, W h s e}{l} + \frac{4\, d^2 s^2 e^2}{l^2}} - \frac{2\, d s e}{l}, \quad . \quad . \quad . \quad (6)$$

Equation (2) differs from Weisbach's and Rankine's on the safe side.

364. EMPIRICAL FORMULAS. General Principles. (1) An empirical formula should be of correct form; (2) the constants in it should be correctly deduced; and (3) the limits within which it is applicable should be stated.

For example, suppose that it were desired to determine the equation of the straight line $A\,B$, Fig. 57. Since the given line is straight, we will assume that the empirical formula is of the form $y = m\,x$. We might find m by measuring the ordinates 1, 2, 3, and place m equal to their mean. If 1, 2, 3, be the numerical values of the respective ordinates, the formula becomes $y = 2\,x$, which gives the line $O\,C$. The mean ordinate to $O\,C$ is equal to the mean ordinate to $A\,B$, but the two are not by any means the

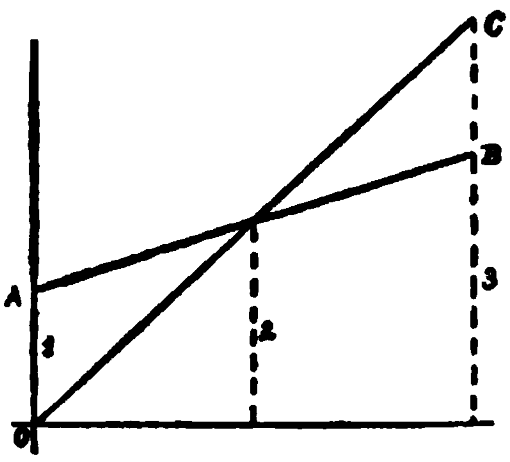


FIG. 57.

* Mechanics of Engineering, 6th ed. (Coxe's Translation), p. 701.

† Civil Engineering, p. 602.

same line. It is evident that this empirical formula is of the wrong form.

For another illustration, assume that some law is correctly represented by the curve $A\ B$, Fig. 58. The form of the empirical formula may be such as to give the curve $C\ D$. These curves coincide exactly at two points, and the mean ordinate to the two is the same. To use a common expression, we may say that, "on the average, the empirical formula agrees exactly with the facts;" but it is, nevertheless, not even approximately true. The constants were not correctly deduced.

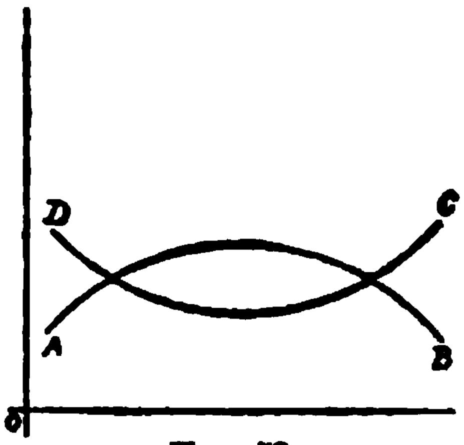


FIG. 58.

Even if of the correct form and correctly deduced, an empirical formula can be safely applied only within the limits of those values from which it was determined. For example, a law may be represented by the curve $A\ B$, Fig. 59. From observations made in the region $C\ E$, the empirical formula has been determined, which gives the curve $C\ E\ D$, which between the limits C and E is all that can be desired, but which is grossly in error between the limits E and D. To use an empirical formula intelligently, it is absolutely necessary that the limits within which it is applicable should be known.

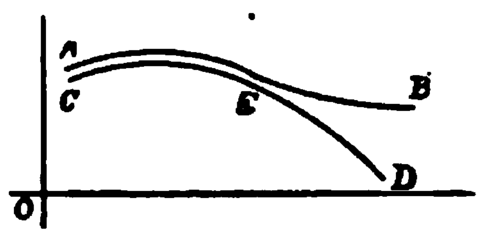


FIG. 59.

Of course, the observations from which the empirical formula was deduced can not be used to test the correctness of the formula; such a procedure can check only the mathematical work of deriving the constants.

Elementary as the preceding principles are, many empirical formulas are worthless owing to a disregard of these conditions in deducing them.

365. Comparison of Empirical Formulas. We will now briefly consider the empirical formulas that are most frequently employed to determine the supporting power of piles.*

Haswell's formula for the dynamic effect of a falling body is † $P = 4.426\ W\ V$, "as deduced from experiments."

The experiments consisted in letting a weight of a few ounces

* For explanation of the nomenclature, see p. 235.

† Haswell's Engineers' and Mechanics' Pocket-Book, p. 419.

fall a few inches upon a coiled spring; and hence the formula is entirely inapplicable to pile driving.

Beaufoy's formula is $P = 0.5003\ W\ V^2$, "as determined by experiment." This formula was deduced under the same conditions as Haswell's, and hence is useless for pile driving. The difference between the formulas is due to the fact that Haswell used only one weight and one spring, and varied the height of the fall, while Beaufoy employed one weight and springs of such relative stiffness as would stop the weight in nearly the same distance for different heights of fall.* Notice that Haswell's, and also Beaufoy's formula, would give the same bearing power for all soils, other things being the same.

Nystrom's formula † is $P = \frac{W^2 h}{(W + w)^2 d}$. In a later book,‡ Nystrom gives the formula $P = \frac{3}{4} \frac{W h}{d}$, assuming that "about 25 per cent. of the energy of the ram is lost by the crushing of the head of the pile." Both of these formulas are roughly approximate, theoretical formulas, although frequently cited as "practical formulas."

Mason's formula § is $P = \frac{W^2 h}{(W + w) d}$. As in the preceding cases, this is frequently referred to as a "practical formula;" but an examination of the original memoir shows that it is wholly a theoretical formula with no pretensions of being anything else. It is also sometimes referred to as having been "tested by a series of experiments;" but apparently the only basis for this is that the piles upon which Fort Montgomery (Rouse's Point, N. Y.) stood from 1846 to 1850 without any sign of failure, when tested by this formula, showed a co-efficient of safety of $3\frac{6}{10}$. The evidence is not conclusive: (1) the factor is large enough to cover a considerable error in the formula; (2) since the formula assumes that all of the energy in the descending ram is expended in overcoming the resistance to penetration, the computed bearing power is too small, and consequently the co-efficient of safety is even greater than as stated;

* Van Nostrand's Engin'g Mag., vol. xvii. p. 325.

† Nystrom's Pocket-Book, p. 158.

‡ New Mechanics, p. 184.

§ Resistance of Piles, J. L. Mason, p. 8; No. 5 of Papers on Practical Engineering, published by the Engineering Department of the U. S. Army.

and (3) it is probably safe to say that after a pile has stood a short time its bearing power is greater than at the moment the driving ceased, owing to the settlement of the earth about it.

*Sander's formula** is $P' = \frac{W\,h}{8\,d}$, in which P' is the *safe* bearing power. This formula was deduced on the assumptions that the energy of the falling weight was wholly employed in forcing the pile into the ground,—*i. e.*, on the assumption that $P\,d = W\,h$, or $P = \frac{W\,h}{d}$,—and that the *safe* load was one eighth of the ultimate supporting power. It is therefore a roughly approximate, theoretical formula.

Notice that, since some of the energy is always lost, $P\,d$, the energy represented by the movement of the pile, must always be less than $W\,h$, the energy of the hammer; hence, P is always less than $\frac{W\,h}{d}$; or, in mathematical language, $P < \frac{W\,h}{d}$. This relation is very useful for determining the greatest possible value of the supporting power. P will always be considerably less than $\frac{W\,h}{d}$; and this difference is greater the lighter the weight, the greater the fall, the softer the material of the pile, or the more the head is bruised. When d is very small, say $\frac{1}{4}$ inch or less, the difference is so great as to make this relation useless.

McAlpine's formula † is $P = 80\,(W + 0.228\,\sqrt{h} - 1)$. It was deduced from experiments made in connection with the construction of the Brooklyn dry-dock. The author of the formula states that it "is applicable only with hammers weighing more than a ton, and only for soil similar to that for which it was deduced, *i. e.*, quicksand." †

Trautwine's formula, ‡ in the nomenclature of page 235, is $P = \frac{52\,W\,\sqrt[3]{h}}{1 + 12\,d}$. It was deduced from the observed supporting power of piles driven in soft soil. Strictly speaking, it is applicable only under conditions similar to those from which it was

* Jour. Frank. Inst., 3d series, vol. xxii. p. —.

† *Ibid.*, vol. lv. pp. 101–02.

‡ Engineer's Pocket-Book, Ed. 1885, p. 648.

deduced; and hence it is inapplicable for hard driving and to piles whose heads are not bruised about the same amount as were the experimental ones. No formula can be accurate which does not, in some way, take cognizance of the condition of the head of the pile. For example, experiments Nos. 3 and 4 of the table on page 246 are the same except in the condition of the heads of the piles, and yet the load supported by the former was $2\frac{1}{2}$ times that supported by the latter. This formula is not applicable to piles driven with a steam hammer, since according to it the energy represented by the sinking of the pile is greater than the total energy in the descending weight. For example, if $W = 1\frac{1}{2}$ tons, $h = 2$ feet, and $d = 1$ inch $= \frac{1}{12}$ of a foot, the formula $P < \frac{W h}{d}$ becomes $P < 36$ tons. Trautwine's formula gives $P = 49$ tons; that is to say, Trautwine's formula makes the supporting power one third more than it would be if *no* energy were lost.

366. *The Author's Empirical Formula.* Certain assumptions and approximations were made in deducing equation (3), page 239. If it is thought not desirable to trust entirely to theory, then the formula

$$P = \sqrt{2\, q\, W h + q^2 d^2} - q\, d \quad . \quad . \quad . \quad . \qquad (7)$$

may be considered as giving only the form which the empirical formula should have. Under this condition q becomes a numerical co-efficient to be determined by experiment, which must be made by driving a pile and measuring d, after which the sustaining power must be determined by applying a direct pressure. The last, or test, blow should be struck on sound wood.

367. Table 29 gives all the experiments on the supporting power of piles for which the record is complete. Unfortunately these experiments do not fulfill the conditions necessary for a proper determination of q in equation (7). It is known that in some of the cases the head of the pile was considerably broomed, and there is internal evidence that this was so in the others.

The data of the following table substituted in equation (7) give values of q from 1.5 to 337, with an average of 130. The range of these results shows the inconsistency of the experiments, and the smallness of the average shows that the last blow was not struck on sound wood. This value of q is of no practical use.

TABLE 29.

DATA OF EXPERIMENTS ON THE SUPPORTING POWER OF PILES.

Number for Reference.	Weight of the Hammer, in Tons.	Height of Fall, in Feet.	Penetration, in Feet.	Observed Supporting Power, in Tons.	Authority.
1	0.455	5	0.031	30.2	Circular of the Office of Chief of Engineers U. S. A., Nov. 12, '81, pp. 2, 3.
2	0.8	36	1.5	7.3	Trautwine's Pocket-Book, ed. 1885, p. 643.
3	1.12	30	0.042	112.0	Jour. Frank. Inst., vol. 55, p. 101.
4	1.1	30	0.042	45.9	Delafield's "Foundations in Compressible Soils," pp. 17, 18;—a pamphlet published by Engineers' Department of U. S. A.
5	0.95	29	0.125	50.0	Trautwine in *Railroad Gazette*, July 8, 1887, p. 453.

368. As confirming the reliability of the *form* of equations (3), (4), and (7), it is interesting to notice that A. C. Hertiz* found, from the records of the driving and afterwards pulling up of nearly 400 piles, the following relation:

$$d = \frac{Wh}{P} - \frac{P}{500},$$

which may be put in the form

$$P = \sqrt{500\,Wh + (250\,d)^2} - 250\,d. \quad . \quad . \quad . \quad . \qquad (8)$$

Equation (8) has exactly the form of equation (3), page 239, although deduced in an entirely different way. The value (250) of the constant q in equation (8) is less than that in equation (4), page 239, which shows that the heads of the piles were broomed. The value of q in equation (8) is greater than that deduced from the data of Table 29, which shows that the piles from which equation (8) was determined were not bruised as much as those in the above table.

369. Supporting Power Determined by Experiment. It is not certain that the bearing power of a pile when loaded with a continued quiescent load will be the same as that during the very short

* Proc. Inst. of C. E., vol. lxiv. pp. 311-15; republished in Van Nostrand's Magazine, vol. xxv. pp. 273-76.

period of the blow. The friction on the sides of the pile will have a greater effect in the former case, while the resistance to penetration of the point will be greater in the latter. This, and the fact that the supporting power of piles sunk by the water-jet can be determined in no other way, shows the necessity of experiments to determine the bearing power under a steady load.

Unfortunately no extended experiments have been made in this direction. We can give only a collection of as many details as possible concerning the piles under actual structures and the loads which they sustain. In this way, we may derive some idea of the sustaining power of piles under various conditions of actual practice.

370. Ultimate Load. In constructing a light-house at Proctorsville, La., in 1856–57, a test pile, 12 inches square, driven 29.5 feet, bore 29.9 tons without settlement, but with 31.2 tons it "settled slowly." The soil, as determined by borings, had the following character: "For a depth of 9 feet there was mud mixed with sand; then followed a layer of sand about 5 feet thick, next a layer of sand mixed with clay from 4 to 6 feet thick, and then followed fine clay. By draining the site the surface was lowered about 6 inches. The pile, by its own weight, sank 5 feet 4 inches." The above load is equivalent to a frictional resistance of 600 lbs. per sq. ft. of surface of pile in contact with the soil. This pile is No. 1 of the table on page 246.

At Philadelphia in 1873, a pile was driven 15 ft. into "soft river mud, and 5 hours after 7.3 tons caused a sinking of a very small fraction of an inch; under 9 tons it sank $\frac{3}{4}$ of an inch, and under 15 tons it sank 5 ft." The above load is equivalent to 320 lbs. per sq. ft. of surface of contact. This pile is No. 2 of the table on page 246.

In the construction of the dock at the Pensacola navy yard, a pile driven 16 feet into clean white sand sustained a direct pressure of 43 tons without settlement, while 45 tons caused it *to rise* slowly; and it required 46 tons to draw a pile that had been driven 16 feet into the sand. This is equivalent to a frictional resistance of 1,900 lbs. per sq. ft. This pile is No. 4 of the table on page 246.

"In the construction of a foundation for an elevator at Buffalo, N. Y., a pile 15 inches in diameter at the large end, driven 18 ft., bore 25 tons for 27 hours without any ascertainable effect. The weight was then gradually increased until the total load on the

pile was 37½ tons. Up to this weight there had been no depression of the pile, but with 37½ tons there was a gradual depression which aggregated ⅝ of an inch, beyond which there was no depression until the weight was increased to 50 tons. With 50 tons there was a further depression of ⅞ of an inch, making the total depression 1½ inches. Then the load was increased to 75 tons, under which the total depression reached 3½ inches. The experiment was not carried beyond this point. The soil, in order from the top, was as follows: 2 ft. of blue clay, 3 ft. of gravel, 5 ft. of stiff red clay, 2 ft. of quicksand, 3 ft. of red clay, 2 ft. of gravel and sand, and 3 ft. of very stiff blue clay. All the time during this experiment there were three pile-drivers at work on the foundation, thus keeping up a tremor in the ground. The water from Lake Erie had free access to the pile through the gravel." * This is equivalent to a frictional resistance of 1,850 lbs. per sq. ft. This is pile No. 5 of the table on page 246.

371. In making some repairs at the Hull docks, England, several hundred sheet-piles were drawn out. They were 12 × 10 inches, driven an average depth of 18 feet in stiff blue clay, and the average force required to pull them was not less than 35.8 tons each. The frictional resistance was at least 1,875 lbs. per sq. ft. of surface in contact with the soil.†

372. Safe Load. The piles under the bridge over the Missouri at Bismarck, Dakota, were driven 32 ft. into the sand, and sustain 20 tons each—equivalent to a frictional resistance of 600 lbs. per sq. ft. The piles at the Plattsmouth bridge, driven 28 ft. into the sand, sustain less than 13½ tons, of which about one fifth is live load,—equivalent to a frictional resistance of 300 lbs. per sq. ft.

At the Hull docks, England, piles driven 16 ft. into "alluvial mud" sustain at least 20 tons, and according to some 25 tons; for the former, the friction is about 800 lbs. per sq. ft. The piles under the Royal Border bridge "were driven 30 to 40 ft. into sand and gravel, and sustain 70 tons each,"—the friction being about 1,400 lbs. per sq. ft.

373. "The South Street bridge approach, Philadelphia, fell by the sinking of the foundation piles under a load of 24 tons each.

* By courtesy of John C. Trautwine, Jr., from private correspondence of John E. Payne and W. A. Haven, engineers in charge.

† Proc. Inst. of C. E., vol. lxiv. pp. 311-15.

They were driven to an absolute stoppage by a 1-ton hammer falling 32 feet. Their length was from 24 to 41 feet. The piles were driven through mud, then tough clay, and into hard gravel."* A possible explanation of the failure of these piles is that they vibrated under the moving load, which allowed the water to work its way down the sides of the piles and thus decrease the bearing power; but it is more probable that the last blow was struck on a broomed head, which would greatly reduce the penetration, and that consequently their supporting power was overestimated. According to Trautwine's formula—the only one of all the preceding which is even approximately applicable to this case—their supporting power was 164 tons.

374. Supporting Power of Screw and Disk Piles. The supporting power depends upon the nature of the soil and the depth to which the pile is sunk. A screw pile "in soft mud above clay and sand" supported 1.8 tons per sq. ft. of blade.† A disk pile in "quicksand" stood 5 tons per sq. ft. under vibrations.‡ Charles McDonald, in constructing the iron ocean-pier at Coney Island, assumed that the safe load upon the flanges of the iron disks sunk into the sand, was 5 tons per sq. ft.; but "many of them really support as much as 6.3 tons per sq. ft. continually and are subject to occasional loads of 8 tons per sq. ft., without causing any settlement that can be detected by the eye."§

375. Factor of Safety. On account of the many uncertainties in connection with piles, a wide margin of safety is recommended by all authorities. The factor of safety ranges from 2 to 12 according to the importance of the structure and according to the faith in the formula employed or the experiment taken as a guide. At best, the formulas can give only the supporting power at the time when the driving ceases. If the resistance is derived mainly from friction, it is probable that the supporting power increases for a time after the driving ceases, since the co-efficient of friction is usually greater after a period of rest. If the supporting power is derived mainly from the resistance to penetration of a stiff substratum, the bearing power for a steady load will probably be smaller than the

* Trans. Am. Soc. of C. E., vol. vii. p. 264.
† Proc. Inst. of C. E., vol. xvii. p. 451.
‡ *Ibid.*, p. 443.
§ Trans. Am. Soc. C. E., vol. viii. p. 236.

force required to drive it, as most materials require a less force to change their form slowly than rapidly. If the soil adjoining the piles becomes wet, the supporting power will be decreased; and vibrations of the structure will have a like effect.

The formulas in use for determining the supporting power of piles are so unreliable, that it is quite impossible to determine the factor of safety for any existing structure with anything like accuracy.

The factor to be employed should vary with the nature of the structure. For example, the abutments of a stone arch should be constructed so that they will not settle at all; but if a railroad pile trestle settles no serious damage is done, since the track can be shimmed up occasionally. In a few cases, a small settlement has taken place in a railroad trestle when the factor of safety was 3 or 4, as computed by equation (4), page 239.

Art. 3. Arrangement of the Foundation.

376. Disposition of the Piles. The length of the piles to be used is determined by the nature of the soil, or the conveniences for driving, or the lengths most easily obtained. The safe bearing power may be determined from the data presented in §§ 370–73, or, better, by driving a test pile and applying equation (4), page 239. Then, knowing the weight to be supported, and having decided upon the length of piles to be used, and having ascertained their safe bearing power, it is an easy matter to determine how many piles are required. Of course, the number of piles under the different parts of a structure should be proportional to the weights of those parts.

If the attempt is made to drive piles too close together, they are liable to force each other up. To avoid this, the centers of the piles should be, at least, 2½ or 3 feet apart. Of course, they may be farther apart, if a less number will give sufficient supporting power, or if a greater area of foundation is necessary to prevent overturning.

When a grillage (§ 380) is to be placed on the head of the piles, great care must be taken to get the latter in line so that the lowest course of grillage timber, in this case called *capping*, may rest squarely upon all the piles of a row. In driving under water, a

convenient way of marking the positions of the piles is to construct a light frame of narrow boards, called a *spider*, in which the position of the piles is indicated by a small square opening. This frame may be held in place by fastening it to the sides of the coffer-dam, or to the piles already driven, or to temporary supports. Under ordinary circumstances, it is reasonably good work if the center of the pile is under the cap. Piles frequently get considerably out of place in driving, in which case they may sometimes be forced back with a block and tackle or a jack-screw. When the heads of the piles are to be covered with concrete, the exact position of the piles is comparatively an unimportant matter.

In close driving, it is necessary to commence at the center of the area and work towards the sides; for if the central ones are left until the last, the soil may become so consolidated that they can scarcely be driven at all.

377. Butt vs. Top Down. According to Rankine* all piles should be driven large end down, having first been sharpened to a point 1½ to 2 times as long as the diameter of the pile. This is at least of doubtful utility. If the pile is supported wholly by friction, then the supporting power will be greater when the small end is down. If the soil is semi-liquid, the buoyancy would be slightly greater when the large end is down; but the buoyancy constitutes but a very small part of the supporting power, and the difference in buoyancy between top and bottom down is still less. If the pile derives its support mainly from a solid substratum, then its bearing power would be greater with the large end down; but, in this case, it should not be sharpened. For close driving, it is frequently recommended that, to prevent the piles from forcing each other up, they should be driven butt end down. Notice, however, that if the soil is non-compressible, as pure sand, or if the piles are driven so close as to compress the soil considerably, it will rise and carry the piles with it, whether they were driven with the big or the little end down. Piles are generally driven small end down, but nevertheless practical experience shows that there are conditions in which it is apparently impossible to drive them in this way, even in comparatively isolated positions. These conditions appear to occur most frequently in swamps, and in connection with quicksand.

* "Civil Engineering," p. 602.

378. Sawing-off the Piles. When piles are driven, it is generally necessary to saw them off either to bring them to the same height, or to get the tops lower than they can be driven, or to secure sound wood upon which to rest the timber platform that carries the masonry. When above water, piles are usually sawed off by hand; and when below, by machinery—usually a circular saw on a vertical shaft held between the leaders of the pile driver or mounted upon a special frame, and driven by the engine used in driving the piles. The saw-shaft is sometimes attached to a vertical shaft held between the leaders by parallel bars, by which arrangement the saw can be swung in the arc of a circle and several piles be cut off without moving the machine. The piles are sometimes sawed off with what is called a pendulum saw, *i.e.*, a saw-blade fastened between two arms of a rigid frame which extends into the water and is free to swing about an axis above. The saw is swung by men pushing on the frame. The first method is the better, particularly when the piles are to be sawed off under mud or silt.

Considerable care is required to get the tops cut off in a horizontal plane. It is not necessary that this shall be done with mathematical accuracy, since if one pile does stand up too far the excess load upon it will either force it down or crush the cap until the other piles take part of the weight. Under ordinary conditions, it is a reasonably good job if piles on land are sawed within half an inch of the same height; and under water, within one inch. When a machine is used on land, it is usually mounted upon a track and drawn along from pile to pile, by which device, after having leveled up the track, a whole row can be sawed off with no further attention. When sawing under water, the depth below the surface is indicated by a mark on the saw-shaft, or a target on the saw-shaft is observed upon with a leveling instrument, or a leveling rod is read upon some part of the saw-frame, etc. In sawing piles off under water, from a boat, a great deal of time is consumed (particularly if there is a current) in getting the boat into position ready to begin work.

Piles are frequently sawed off under 10 to 15 feet of water, and occasionally under 20 to 25 feet.

379. Finishing the Foundation. There are two cases: (1) when the heads of the piles are not under water; and (2) when they are under water.

1. When the piles are not under water there are again two cases: (*a*) when a timber *grillage* is used; and (*b*) when *concrete* alone is used.

2. When the piles are sawed off under water, the timber structure (in this case called a *crib*) which intervenes between the piles and the masonry is put together first, and then sunk into place. The construction is essentially the same as when the piles are not under water, but differs from that case in the manner of getting the timber into its final resting place. The methods of constructing foundations under water, including that by the use of timber cribs, will be discussed in Art. 2 of the next chapter.

380. Piles and Grillage. This is a stout frame of one or more courses of timber drift-bolted or pinned to the tops of the piles and to each other, upon which a floor of thick boards is placed to receive the bottom courses of masonry. For illustrated examples, see Fig. 84, page 362, Fig. 86, page 380, and Fig. 90, page 386.

The timbers which rest upon the heads of the piles, called *caps*, are usually about 1 foot square, and are fastened by boring a hole through each and into the head of the pile and driving into the hole a plain rod or bar of iron having about 25 per cent. larger cross section than the hole.

381. These rods are called *drift-bolts*, and are usually either a rod 1 inch in diameter (driven into a $\frac{3}{4}$-inch auger hole), or a bar 1 inch square (driven into a $\frac{7}{8}$-inch hole). Formerly jag-bolts, or rag-bolts, *i. e.*, bolts whose sides were jagged, or barbed, were used for this and similar purposes; but universal experience shows that smooth rods hold much the better. In some experiments made at the Poughkeepsie bridge (§ 414), it was found that a 1-inch rod driven into a $\frac{15}{16}$-inch hole in hemlock required on the average a force of $2\frac{1}{8}$ tons per linear foot of rod to withdraw it; and a 1-inch rod driven into a $\frac{3}{4}$-inch hole in white or Norway pine required 5 tons per linear foot of rod to withdraw it. The old-style jag-bolt was square because it was more easily barbed; and probably this is the reason why square drift-bolts are now more common. Another advantage of the round drift-bolt, over the square one, is that the latter does not cut or tear the wood as much as the former. The ends of the rods should be slightly rounded with a hammer.

Transverse timbers are put on top of the caps and drift-bolted to them. Old bridge-timbers, timbers from false works, etc., are

frequently used, and are ordinarily as good for this purpose as new. As many courses may be added as is necessary, each perpendicular to the one below it. The timbers of the top course are laid close together, or, as before stated, a floor of thick boards is added on top to receive the masonry.

This form of construction is very common in the foundations of bridge abutments. Of course no timber should be used in a foundation, except where it will always be wet.

382. **Piles and Concrete.** A thick layer of concrete, resting partly on the heads of the piles and partly on the soil between them, is frequently employed instead of the timber grillage as above. Objection is sometimes made to the platform (§ 380) as a bed for a foundation that, owing to the want of adhesion between wood and mortar, the masonry might slide off from the platform if any unequal settling should take place. To obviate this, the concrete is frequently substituted for the grillage and platform.

However, there is but slight probability that a foundation will ever fail on account of the masonry's sliding on timber, since, ordinarily, this could take place only when the horizontal force is nearly half of the downward pressure.* This could occur only with dams, retaining walls, or bridge abutments, and rarely, if ever, with these. One of the fundamental principles of all masonry construction is to build the courses perpendicular to the line of pressure, which condition alone would prevent slipping. Any possibility of slipping can be prevented also by omitting one or more of the timbers in the top course—the omitted timbers being perpendicular to the direction of the forces tending to produce sliding,—or by building the top of the grillage in the form of steps, or by driving drift-bolts into the platform and leaving their upper ends projecting.

Although the use of concrete, as above, may not be necessary to prevent sliding, it adds materially to the supporting power of the foundation; it utilizes the bearing power of the soil between the piles as well as the supporting power of the piles themselves, which is a very important consideration in soft soils. Another advantage of this form of construction is that the concrete can be laid without exhausting the water or sawing off the piles. Frequently

* See Table 36, page 315.

concrete can also be used advantageously in connection with timber grillage to pack in around the timbers.

383. Lateral Yielding. Notice that, although the masonry may not slide off from the timber platform (§ 382), the foundation may yield laterally by the piles themselves being pushed over. If the piles reach a firm subsoil, it will help matters a little to remove the upper and more yielding soil from around the tops of the piles and fill in with broken stone; or a wall of piles may be driven around the foundation—at some distance from it,—and timber braces be placed between the wall of piles and the foundation. When the foundation can not be buttressed in front, the structure may be prevented from moving forward by rods which bear on the face of the wall and are connected with plates of iron or blocks of stone imbedded in the earth at a distance behind the wall (see § 551), or the thrust of the earth against the back of the wall may be decreased by supporting the earth immediately behind the foundation proper upon a grillage and platform resting on piles, or the same result may be attained by constructing relieving arches against the back of the wall (see § 552).

384. Cushing's Pile Foundation. The desire to utilize the cheapness and efficiency of ordinary piles as a foundation for bridge piers and at the same time secure greater durability than is possible with piles alone, led to the introduction of what is known as Cushing's pile foundation, first used in 1868, at India Point, Rhode Island. It consists of square timber piles in intimate contact with each other, forming a solid mass of bearing timber. Surrounding the pile cluster is an envelope of cast or wrought iron, sunk in the mud or silt only enough to protect the piles, all voids between piles and cylinders being filled with hydraulic concrete.

Several such foundations have been used, and have proved satisfactory in every respect. The only objection that has ever been urged against them is that the piles may rot above the water line. If they do rot at all, it will be very slowly; and time alone can tell whether this is an important objection.

In making a foundation according to the Cushing system, the piles may be driven first and the cylinder sunk over them, or the piles can be driven inside the cylinder after a few sections are in place. In the latter case, however, the cylinders may be subjected to undue strains and to subsequent damage from shock and

vibration; and besides, the sawing off of the piles would be very difficult and inconvenient, and they would have to be left at irregular heights and with battered tops. On the other hand, if the piles are driven first, there is danger of their spreading and thereby interfering with the sinking of the cylinder.

The special advantages of the Cushing piers are: (1) cheapness, (2) ability to resist scour, (3) small contraction of the water way, and (4) rapidity of construction.

385. Example. The railroad bridge over the Tenas River, near Mobile, rests on Cushing piers. There are thirteen, one being a pivot pier. Each, excepting the pivot pier, is made of two cast-iron cylinders, 6 feet in exterior diameter, located 16 feet between centers. The cylinders were cast in sections 10 feet long, of metal 1½ inches thick, and united by interior flanges 2 inches thick and 3 inches wide. The sections are held together by 40 bolts, each 1¼ inches in diameter. The lower section in each pier was provided with a cutting-edge, and the top section was cast of a length sufficient to bring the pier to its proper elevation.

The pivot pier is composed of one central cylinder 6 feet in diameter, and six cylinders 4 feet in diameter arranged hexagonally. The radius of the pivot circle, measuring from the centers of cylinders, is 12½ feet. Each cylinder is capped with a cast-iron plate 2½ inches thick, secured to the cylinder with twenty 1-inch bolts.

The piles are sawed pine, not less than 10 inches square at the small end. They were driven first, and the cylinder sunk over them. In each of the large cylinders, 12 piles, and in each of the smaller cylinders, 5 piles, were driven to a depth not less than 20 feet below the bed of the river. The piles had to be in almost perfect contact for their whole length, which was secured by driving their points in contact as near as possible, and then pulling their tops together and holding them by 8 bolts 1½ inches in diameter. In this particular bridge the iron cylinders were sunk to a depth not less than 10 feet below the river bed; but usually they are not sunk more than 3 to 7 feet. The piles were cut off at low water, the water pumped out of the cylinder, and the latter then filled to the top with concrete.

CHAPTER XII.

FOUNDATIONS UNDER WATER.

386. The class of foundations to be discussed in this chapter could appropriately be called Foundations for Bridge Piers, since the latter are about the only ones that are laid under water. In this class of work two difficulties have to be overcome, both of which require great resources and care on the part of the engineer. The first is found in the means to be used in preparing the bed of the foundation, and the second in preserving it from the scouring action of the water.

Preventing the undermining of the foundation is generally not a matter of much difficulty. In quiet water or in a sluggish stream but little protection is required; in which case it is sufficient to deposit a mass of loose stone, or riprap, around the base of the pier. If there is danger of the riprap's being undermined, the layer must be extended farther from the base, or be made so thick that, if undermined, the stone will fall into the cavity and prevent further damage. A willow mattress sunk by placing stones upon it is an economical and efficient means of protecting a structure against scour. A pier may be protected also by inclosing it with a row of piles and depositing loose rock between the pier and the piles. In minor structures the foundation may be protected by driving sheet piles around it.

If a large quantity of stone be deposited around the base of the pier, the velocity of the current, and consequently its scouring action, will be increased. Such a deposit is also an obstruction to navigation, and therefore is seldom permitted. In many cases the only absolute security is in sinking the foundation below the scouring action of the water. The depth necessary to secure this adds to the difficulty of preparing the bed of the foundation.

387. The principal difficulty in laying a foundation under water consists in excluding the water. If necessary, masonry can be laid under water by divers; but this is very expensive and is rarely resorted to.

There are five methods in use for laying foundations under water: (1) the method of excluding the water from the bed of the foundation by the use of a coffer-dam; (2) the method of founding the pier, without excluding the water, by means of a timber crib surmounted by a water-tight box in which the masonry is laid; (3) the method of sinking iron tubes or masonry wells to a solid substratum by excavating inside of them; (4) the method in which the water is excluded by the presence of atmospheric air; and (5) the method of freezing a wall of earth around the site, inside of which the excavation can be made and the masonry laid. These several methods will be discussed separately in the order named.

ART. 1. THE COFFER-DAM PROCESS.

388. A *coffer-dam* is an inclosure from which the water is pumped and in which the masonry is laid in the open air. This method consists in constructing a coffer-dam around the site of the proposed foundation, pumping out the water, preparing the bed of the foundation by driving piles or otherwise, and laying the masonry on the inside of the coffer-dam. After the masonry is above the water the coffer-dam can be removed.

389. CONSTRUCTION OF THE DAM.* The construction of coffer-dams varies greatly. In still, shallow water, a well-built bank of clay and gravel is sufficient. If there is a slow current, a wall of bags partly filled with clay and gravel does fairly well; a row of cement barrels filled with gravel and banked up on the outside has also been used. If the water is too deep for any of the above methods, a single or double row of sheet piles may be driven and banked up on the outside with a deposit of impervious soil sufficient to prevent leaking. If there is much of a current, the puddle on the outside will be washed away; or, if the water is deep, a large quantity of material will be required to form the puddle-wall; and hence the preceding methods are of limited application.

390. The ordinary method of constructing a coffer-dam in deep water or in a strong current is shown in Fig. 60. The area to be inclosed is first surrounded by two rows of ordinary piles, *m, m*. On the outside of the main piles, a little below the top, are bolted two

* See also § 317, page 214.

longitudinal pieces, *w*, *w*, called wales; and on the inside are fastened two similar pieces, *g*, *g*, which serve as guides for the sheet piles, *s*, *s*, while being driven. A rod, *r*, connects the top of the opposite main piles to prevent spreading when the puddle is put in. The timber, *t*, is put on primarily to carry the footway, *f*, and is sometimes notched over, or otherwise fastened to, the pieces *w*, *w* to prevent the puddle space from spreading. *b* and *b* are braces extending from one side of the coffer-dam to the other. These braces are put in position successively from the top as the water is pumped

FIG. 60.

out; and as the masonry is built up, they are removed and the sides of the dam braced by short struts resting against the pier.

The resistance to overturning is derived principally from the main piles, *m*, *m*. The distance apart and also the depth to which they should be driven depends upon the kind of bottom, the depth of water, and the danger from floating ice, logs, etc. Rules and formulas are here of but little use, judgment and experience being the only guides. The distance between the piles in a row is usually from 4 to 6 feet.

The dimensions of the sheet piles (§ 329) employed will depend upon the depth and the number of longitudinal waling pieces used. Two thicknesses of ordinary 2-inch plank are generally employed. Sometimes for the deeper dams, the sheet piles are timbers 10 or 12 inches square.

The thickness of the dam will depend upon (1) the width of gangway required for the workmen and machinery, (2) the thickness re-

quired to prevent overturning, and (3) the thickness of puddle necessary to prevent leakage through the wall. The thickness of shallow dams will usually be determined by the first consideration; but for deep dams the thickness will be governed by the second or third requirement. If the braces, *b*, *b*, are omitted, as is sometimes done for greater convenience in working in the coffer-dam, then the main piles, *m*, *m*, must be stronger and the dam wider in order to resist the lateral pressure of the water. A rule of thumb frequently used for this case is: "For depths of less than 10 feet make the width 10 feet, and for depths over 10 feet give an additional thickness of 1 foot for each additional 3 feet of wall." Trautwine's rule is to make the thickness of the puddle-wall three fourths of its height; but in no case is the wall to be less than 4 feet thick. If the coffer-dam is well braced across the inclosed area, the puddle-wall may vary from 3 feet for shallow depths to 10 feet for great depths; the former width has been successfully employed for depths of 18 to 20 feet, although it is considerably less than is customary.

The puddle-wall should be constructed of impervious soil, of which gravelly clay is best. It is a common idea that clay alone, or clay and fine sand, is best. With pure clay, if a thread of water ever so small finds a passage under or through the puddle, it will steadily wear a larger opening. On the other hand, with gravelly clay, if the water should wash out the clay or fine sand, the larger particles will fall into the space and intercept first the coarser sand, and next the particles of loam which are drifting in the current of water; and thus the whole mass puddles itself better than the engineer could do it with his own hands. An embankment of gravel is comparatively safe, and becomes tighter every day. While a clay embankment may be tighter at first than a gravelly one, it is always liable to breakage. Before putting in the puddling, all soft mud and loose soil should be removed from between the rows of sheet piles. The puddling should be deposited in layers, and compacted as much as is possible without causing the sheet piles to bulge so much as to open the joints.

391. Coffer-dams are sometimes constructed by building a strong crib, and sinking it. The crib may be composed either of uprights framed into caps and sills and covered on the outside with tongued and grooved planks, or of squared timbers laid one on top of the other, log-house fashion, and well calked. The outer uprights are

braced against the inside uprights and sills to prevent crushing inwards. This crib may be built on land, launched, towed to its final place, and sunk by piling stones on top or by throwing them into cells of the crib-work which are boarded up for that purpose. The bottom of the stream may be leveled off to receive the crib by dredging, or the dam may be made tight at the bottom by driving sheet piles around it. The crib must be securely bolted together (see § 381) vertically, or the buoyancy of the water will lift off the upper courses.

A movable coffer-dam is sometimes constructed in the same general way, except that it is made in halves to allow of removal from around the finished pier. The two halves are joined together by fitting timbers between the projecting courses of the crib, and then passing long bolts vertically through the several courses. Some of the compartments are made water-tight to facilitate the movement of the crib from place to place.*

Coffer-dams are also built by sinking an open crib, similar to the above, and then sheeting it on the outside by driving piles around it after it is sunk. For shallow depths, this method is very efficient.

392. Sometimes two coffer-dams are employed, one inside of the other, the outer one being used to keep out the water, and the inner one to keep the soft material from flowing into the excavation. The outer one may be constructed in any of the ways described above. The inner one is usually a frame-work sheeted with boards, or a crib of squared timbers built log-house fashion with tight joints. The inner crib is sunk (by weighting it with stone) as the excavation proceeds. The advantages of the use of the inner crib are (1) that the coffer-dam is smaller than if the saturated soil were allowed to take its natural slope from the inside of the dam to the bottom of the excavation; (2) the space between the crib and the dam can be kept full of impervious material in case of any trouble with the outside dam; (3) the feet of the sheet piling are always covered, which lessens the danger of undermining or of an inflow of water and mud under the dam; and (4) it also reduces to a minimum the material to be excavated.

393. Iron has been used in a few instances as a sheeting for coffer-dams. Plates are riveted together to form the walls, and stayed

* For an illustrated example, see Proc. Engineer's Club of Philadelphia, vol. iv. No. 4.

on the inside by horizontal rings made of angle iron. Wood is cheaper and more easily wrought, and therefore generally preferred.

394. Leakage. A serious objection to the use of coffer-dams is the difficulty of preventing leakage under the dam. One of the simplest devices to prevent this is to deposit a bank of gravel around the outside of the dam; then if a vein of water escapes below the sheet piling, the weight of the gravel will crush down and fill the hole before it can enlarge itself enough to do serious damage. If the coffer-dam is made of crib-work, short sheet piles may be driven around the bottom of it; or hay, willows, etc., may be laid around the bottom edge, upon which puddle and stones are deposited; or a broad flap of tarpaulin may be nailed to the lower edge of the crib and spread out loosely on the bottom, upon which stones and puddle are placed. A tarpaulin is frequently used when the bottom is very irregular,—in which case it would cost too much to level off the site of the dam; and it is particularly useful where the bottom is rocky and the sheet piles can not be driven.

When the bed of the river is rock, or rock covered with but a few feet of mud or loose soil, a coffer-dam only sufficiently tight to keep out the mud is constructed. The mud at the bottom of the inclosed area is then dredged out, and a bed of concrete deposited under the water (§ 154). Before the concrete has set, another coffer-dam is constructed, inside of the first one, the latter being made water-tight at the bottom by settling it into the concrete or by driving sheet piles into it. However, the better and more usual method is to sink the masonry upon the bed of concrete by the method described in Art. 2 (pages 266–71).

It is nearly impossible to prevent considerable leakage, unless the bottom of the crib rests upon an impervious stratum or the sheet piles are driven into it. Water will find its way through nearly any depth or distance of gravelly or sandy bottom. Trying to pump a river dry through the sand at the bottom of a coffer-dam is expensive. However, the object is not to prevent all infiltration, but only to so reduce it that a moderate amount of bailing or pumping will keep the water out of the way. Probably a coffer-dam was never built that did not require considerable pumping; and not infrequently the amount is very great,—so great, in fact, as to make it clear that some other method of constructing the foundation should have been chosen.

Seams of sand are very troublesome. Logs or stones under the edge of the dam are also a cause of considerable annoyance. It is sometimes best to dredge away the mud and loose soil from the site of the proposed coffer-dam; but, when this is necessary, it is usually better to construct the foundation without the use of a coffer-dam,—see Art. 2 of this chapter (page 266). Coffer-dams should be used only in very shallow water, or when the bottom is clay or some material impervious to water.

395. Pumps. In constructing foundations, it is frequently necessary to do considerable bailing or pumping. The method to be employed in any particular case will vary greatly with the amount of water present, the depth of the excavation, the appliances at hand, etc. The pumps generally used for this kind of work are (1) the ordinary wooden hand-pump, (2) the steam siphon, (3) the pulsometer, and (4) the centrifugal pump. Rotary and direct-acting steam pumps are not suitable for use in foundation work, owing to the deleterious effect of sand, etc., in the water to be pumped.

1. *Hand Power.* When the lift is small, water can be bailed out faster than it can be pumped by hand; but the labor is proportionally more fatiguing. The ordinary hand foundation-pump consists of a straight tube at the bottom of which is fixed a common flap valve, and in which works a piston carrying another valve. The tube is either a square wooden box or a sheet-iron cylinder,—usually the latter, since it is lighter and more durable. The pump is operated by applying the power directly to the upper end of the piston-rod, the pump being held in position by stays or ropes. There are more elaborate foundation-pumps on the market.

2. The *steam siphon* is the simplest of all pumps, since it has no movable parts whatever. It consists essentially of a discharge pipe—open at both ends—through the side of which enters a smaller pipe having its end bent up. The lower end of the discharge pipe dips into the water; and the small pipe connects with a steam boiler. The steam, in rushing out of the small pipe, carries with it the air in the upper end of the discharge pipe, thus tending to form a vacuum in the lower end of that pipe; the water then rises in the discharge pipe and is carried out with the steam. Although it is possible by the use of large quantities of steam to raise small quantities of water to a great height, the steam siphon is limited practically to lifting water only a few feet. Its cheapness and simplicity

are recommendations in its favor, and its efficiency is not much less than that of other forms of pumps. A common form of the steam siphon resembles, in external appearance, the Eads sand-pump represented in Fig. 66 (page 293).

3. The *pulsometer* is an improved form of the steam siphon. It may properly be called a steam pump which dispenses with all movable parts except the valves. The height to which it may lift water is practically unlimited.

4. The *centrifugal pump** consists of a set of blades revolving in a short cylindrical case which connects at its center with a suction (or inlet) pipe, and at its circumference with a discharge pipe. The blades being made to revolve rapidly, the air in the case is carried outward by the centrifugal force, tending to produce a vacuum in the suction pipe; the water then enters the case and is discharged likewise. The distance from the water to the pump is limited by the height to which the ordinary pressure of the air will raise the water;† but the height to which a centrifugal pump can lift the water is limited only by the velocity of the outer ends of the revolving blades. When a quick application with a discharge of large quantities of water is the most important consideration, the centrifugal pump is of great value. Since there are no valves in action while the pump is at work, the centrifugal pump will allow sand and large gravel—in fact almost anything that can enter between the arms—to pass. Pumps having a 6-inch to 10-inch discharge pipe are the sizes most frequently used in foundation work.

396. Preparing the Foundation. After the water is pumped out, the bed of the foundation may be prepared to receive the masonry by any of the processes described in §§ 283–91, which see. Ordinarily the only preparation is to throw out, usually with hand shovels, the soft material. The masonry may be started directly upon the hard substratum, or upon a timber grillage resting on the soil (§§ 309–10) or on piles (§ 380).

397. Cost. It is universally admitted that estimates for the cost of foundations under water are very unreliable, and none are more so than those contemplating the use of a coffer-dam. The estimates of the most experienced engineers frequently differ greatly

* Frequently, but improperly, called a *rotary* pump.

† Some forms of centrifugal pumps must be immersed in the liquid to be raised.

from the actual cost. The difficulties of the case have already been discussed (§ 394).

For the cost of piles and driving, see §§ 346–54. The timber will cost, according to locality, anywhere from $15 to $25 per thousand feet, board measure. The cost of labor in placing the timber can not be given, since it varies greatly with the design, size, depth, etc. The iron in drift-bolts, screw-bolts, and spikes, is usually estimated at 3½ to 5 cents per pound in place. Excavation in coffer-dams frequently costs as high as $1 to $1.50 per cubic yard, including the necessary pumping.

398. Example. The following example is interesting as showing the cost under the most favorable conditions. The data are for a railroad bridge across the Ohio River at Point Pleasant, W. Va.* There were three 250-foot spans, one 400-foot, and one 200-foot. There were two piers on land and four in the water; and all extended about 90 feet above low water. The shore piers were founded on piles—driven in the bottom of a pit—and a grillage, concrete being rammed in around the timber. The foundations under water were laid by the use of a double coffer-dam (§ 392). The water was 10 feet deep; and the soil was 3 to 6 feet of sand and gravel resting on dry, compact clay. The foundations consisted of a layer of concrete 1 foot thick on the clay, and two courses of timbers. The quantities of materials in the six foundations, and the total cost, are as follows:

Pine timber in cribs inside of coffer-dams, and in foundations,	278,210	ft. B.M.
Oak timber in coffer-dams, main and sheet piling...........	244,412	" "
Poplar timber in coffer-dams...........................	8,597	" "
Round piles in foundation and coffer-dams.................	18,571	lin. ft.
Excavation in foundations..............................	4,342	cu. yds.
Concrete " "	649	" "
Riprap..	997	" "

The total cost of foundations, including labor of all kinds, derricks, barges, engines, pumps, iron, tools, ropes, and everything necessary for the rapid completion of the work, was $64,652.62.

In the construction of the bridge over the Missouri River, near Plattsmouth, Neb., a concrete foundation 49 feet long, 21 feet wide, and 32 feet deep, laid on shore, the excavation being through clay, bowlders, shale, and soapstone, to bed-rock (32 feet below

* *Engineering News*, vol. xiii. p. 338.

surface of the water), cost $39,607.23, or $42.81 per yard for the concrete laid.*

399. For the relative cost of foundations, see Art. 6, page 307.

400. **Conclusion.** Uncertainty as to what trouble and expense a coffer-dam will develop usually causes engineers to choose some other method of laying the foundations for bridge piers. Coffer-dams are applicable in shallow depths only; hence one objection to founding bridge piers by this process, particularly in rivers subject to scour or liable to ice gorges, is the danger of their being either undermined or pushed off the foundation. When founded in mud or sand, the first mode of failure is most to be feared. This danger is diminished by the use of piles or large quantities of riprap; but such a foundation needs constant attention. When founded on rock, there is a possibility of the piers being pushed off the foundation; for, since it is not probable that the coffer-dam can be pumped perfectly dry and the bottom cleaned before laying the masonry or depositing the concrete, there is no certainty that there is good union between the base of the pier and the bed-rock.

Coffer-dams are frequently and advantageously employed in laying foundations in soft soils not under water, as described in §§ 316–21 (pages 214–15).

Art. 2. The Crib and Open-Caisson Process.

401. **Definitions.** Unfortunately there is an ambiguity in the use of the word *caisson*. Formerly it always meant a strong, water-tight box having vertical sides and a bottom of heavy timbers, in which the pier is built and which sinks, as the masonry is added, until its bottom rests upon the bed prepared for it. With the introduction of the compressed-air process, the term caisson was applied to a strong, water-tight box—open at the bottom and closed at the top—upon which the pier is built, and which sinks to the bottom as the masonry is added. At present, the word caisson generally has the latter meaning. In the pneumatic process, a water-tight box—open at the top—is usually constructed on the roof of the working chamber ("pneumatic chamber"), inside of which the masonry is built; this box also is called a caisson. The caisson

* Exclusive of cost of buildings, tools, and engineering expenses. These items amounted to 6 per cent. of the total cost of the entire bridge.

open at the bottom is sometimes called an *inverted* caisson, and the one open at the top an *erect* caisson. The latter when built over an inverted, or pneumatic, caisson, is sometimes called a coffer-dam. For greater clearness the term *caisson* will be used for the inverted, or pneumatic, caisson ; and the erect caisson, which is built over a pneumatic caisson, will be called a *coffer-dam*. A caisson employed in other than pneumatic work will be called an *open caisson*.

402. PRINCIPLE. This method of constructing the foundation consists in building the pier in the interior of an open caisson, which sinks as the masonry is added and finally rests upon the bed prepared for it. The masonry usually extends only a foot or two below extreme low water, the lower part of the structure being composed of timber crib-work, called simply a *crib*. The open caisson is built on the top of the crib, which is practically only a thick bottom for the box. The timber is employed because of the greater facility with which it may be put into place, as will appear presently. Timber, when always wet, is as durable as masonry ; and ordinarily there is not much difference in cost between timber and stone.

If the soil at the bottom is soft and unreliable, or if there is danger of scour in case the crib were to rest directly upon the bottom, the bed is prepared by dredging away the mud (§ 407) to a sufficient depth or by driving piles which are afterwards sawed off (§ 378) to a horizontal plane.

403. CONSTRUCTION OF THE CAISSON. The construction of the caisson differs materially with its depth. The simplest form is made by erecting studding by toe-nailing or tenoning them into the top course of the crib and spiking planks on the outside. For a caisson 6 or 8 feet deep, which is about as deep as it is wise to try with this simple construction, it is sufficient to use studding 6 inches wide, 3 inches thick, and 6 to 8 feet long, spaced 3 feet apart, mortised and tenoned into the deck course of the crib. The sides and floor (the upper course of the crib) should be thoroughly calked with oakum. The sides may be braced from the masonry as the sinking proceeds. When the crib is grounded and the masonry is above the water, the sides of the box or caisson are knocked off.

When the depth of water is more than 8 to 10 feet, the caisson is constructed somewhat after the general method shown in Fig. 61. The sides are formed of timbers framed together and a covering of thick planks on the outside. The joints are carefully calked to

make the caisson water-tight. In deep caissons, the sides can be built up as the masonry progresses, and thus not be in the way of the masons. The sides and bottom are held together only by the heavy vertical rods; and after the caisson has come to a bearing upon the soil and after the masonry is above the water, the rods are detached and the sides removed, the bottom only remaining as a part of the permanent structure.

For an illustration of the form of caisson employed in sinking a foundation by the compressed-air process, see Plate I.

404. The caisson should be so contrived that it can be

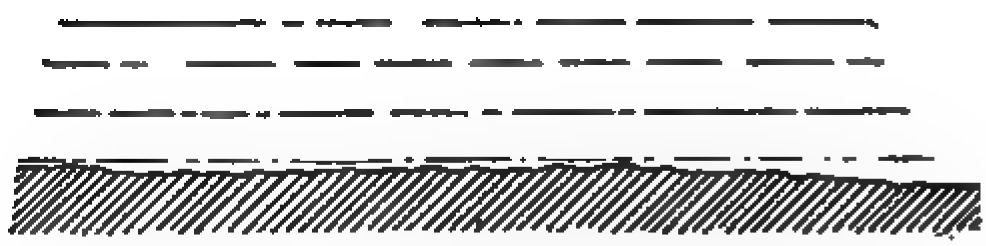


FIG. 61.

grounded, and afterwards raised in case the bed is found not to be accurately leveled. To effect this, a small sliding gate is sometimes placed in the side of the caisson for the purpose of filling it with water at pleasure. By means of this gate, the caisson can be filled and grounded; and by closing the gate and pumping out the water, it can be set afloat. The same result can be accomplished by putting on and taking off stone.

Since the caisson is a heavy, unwieldy mass, it is not possible to control the exact position in which it is sunk; and hence it should be larger than the base of the proposed pier, to allow for a little adjustment to bring the pier to the desired location. The margin to

be allowed will depend upon the depth of water, size of caisson, facilities, etc. A foot all round is probably none too much under favorable conditions, and generally a greater margin should be allowed.

405. Construction of the Crib. The crib is a timber structure below the caisson, which transmits the pressure to the bed of the foundation. A crib is essentially a grillage (see § 309 and § 380) which, instead of being built in place, is first constructed and then sunk to its final resting place in a single mass. A crib is usually thicker, *i. e.*, deeper, than the grillage. If the pressure is great, the crib is built of successive courses of squared timbers in contact; but if the pressure is small, it is built more or less open. In either case, if the crib is to rest upon a soft bottom, a few of the lower courses are built open so that the higher portions of the bed may be squeezed into these cells, and thus allow the crib to come to an even bearing. If the crib is to rest upon an uneven rock bottom, the site is first leveled up by throwing in broken stone. If the bottom is rough or sloping, the lower courses of the crib are sometimes made to conform to the bottom as nearly as possible, as determined from soundings. This method requires care and judgment to prevent the crib from sliding off from the inclined bed, and should be used with great caution, if at all.

The crib is usually built afloat. Owing to the buoyancy of the water, about one third of a crib made wholly of timber would project above the water, and would require an inconveniently large weight to sink it; therefore, it is best to incorporate considerable stone in the crib-work. If the crib is more or less open, this is done by putting a floor into some of the open spaces or pockets, which are then filled with stone. If the crib is to be solid, about every third timber is omitted and the space filled with broken stone.

The timbers of each course should be securely drift-bolted (§ 381) to those of the course below to prevent the buoyancy of the upper portion from pulling the crib apart, and also to prevent any possibility of the upper part's sliding on the lower.

406. Timber in Foundations. The free use of timber in foundations is the chief difference between American and European methods of founding masonry in deep water. The consideration that led to its introduction in foundations was its cheapness. Many of the more important bridges built some years ago rest upon crib-

work of round logs notched at their intersection and secured by drift-bolts. At present, cribs are always built of squared timber. As a rule, there is now but very little difference between the cost of timber and masonry in foundations. The principal advantage in the use of the timber in foundations under water is the facility with which it is put into position. Soft wood or timber which in the air has comparatively little durability, is equally as good for this purpose as the hard woods. It has been conclusively proved that any kind of timber will last practically forever, if completely immersed in water.

407. **Excavating the Site.** When a pier is to be founded in a sluggish stream, it is only necessary to excavate a hole in the bed of the stream, in which the crib (or the bottom of the caisson) may rest. The excavation is usually made with a dredge, any form of which can be employed. The dipper dredge is the best, but the clam-shell or the endless chain and bucket dredge are sometimes used. If the bottom is sand, mud, or silt, the soil may be removed (1) by pumping it with the water through an ordinary centrifugal pump (§ 395),—the suction hose of which is kept in contact with, or even a little below, the bottom,—or (2) by the Eads sand-pump (§ 448). With either of these methods of excavating, a simple frame or light coffer-dam may be sunk to keep part of the loose soil from running into the excavation.

408. If the stream is shallow, the current swift, and the bottom soft, the site may be excavated or scoured out by the river itself. To make the current scour, construct two temporary wing-dams, which diverge up stream from the site of the proposed pier. The wings can be made by driving stout stakes or small piles into the bed of the stream, and placing solid panels—made by nailing ordinary boards to light uprights—against the piles with their lower edge on the bottom. The wings concentrate the current at the location of the pier, increase its velocity, and cause it to scour out the bed of the stream. This process requires a little time, usually one to three days, but the cost of construction and operation is comparatively slight.

When the water is too deep for the last method, it is sometimes possible to suspend the caisson a little above the bed of the stream, in which case the current will remove the sand and silt from under it. At the bridge over the Mississippi at Quincy, Ill., a hole 10 feet

deep was thus scoured out. If the water is already heavily charged with sediment, it may drop the sediment on striking the crib and thus fill up instead of scour out. Notwithstanding the hole is liable to be filled up by the gradual action of the current or by a sudden flood, before the crib has been placed in its final position, this method is frequently more expeditious and less expensive than using a coffer-dam.

409. If the crib should not rest squarely upon the bottom, it can sometimes be brought down with a water-jet (§ 343) in the hands of a diver. However, the engineer should not employ a diver unless absolutely necessary, as it is very expensive.

410. If the soft soil extends to a considerable depth, or if the necessary spread of foundation can not be obtained without an undesirable obstruction of the channel, or if the bottom is liable to scour, then piles may be driven, upon which the crib or caisson may finally rest. Before the introduction of the compressed-air process, this was a very common method of founding bridge piers in our western rivers; and it is still frequently employed for small piers. The method of driving and sawing off the piles has already been described—see Chapter XI.

The mud over and around the heads of the piles may be sucked off with a pump, or it may be scoured out by the current (§ 408). The attempt is sometimes made to increase the bearing power of the foundation by filling in between the heads of the piles with broken stone or concrete; but this is not good practice, as the stone does but little good, is difficult to place, and is liable to get on top of the piles and prevent the crib from coming to a proper bearing.

ART. 3. DREDGING THROUGH WELLS.

411. A timber crib is frequently sunk by excavating the material through apartments left for that purpose, thus undermining the crib and causing it to sink. Hollow iron cylinders, or wells of masonry with a strong curb, or ring, of timber or iron beneath them, are sunk in the same way.

This method is applicable to foundations both on dry land and under water. It is also sometimes employed in sinking shafts in tunneling and mining.

412. EXCAVATORS. The soil is removed from under the crib

with a clam-shell dredge, or with an endless chain and bucket dredge, or with the Eads sand-pump, or, for small jobs, with the sand-pump employed in driving artesian wells.

The clam-shell dredge consists of the two halves of a hemispherical shell, which rotate about a horizontal diameter; the edges of the shell are forced into the soil by the weight of the machine itself, and the pull upon the chain to raise the excavator draws the two halves together, thus forming a hemispherical bucket which incloses the material to be excavated. The Morris and Cumming dredge consists of two quadrants of a short cylinder, hinged and operated similarly to the above. The Milroy dredge (represented at A in Fig. 62, page 274) appears to have the preference for this kind of work. It consists of a frame from which are suspended a number of spherical triangular spades which are forced vertically into the ground by their own weight; the pull upon the excavator to lift it out of the mud draws these triangles together and encloses the earth to be excavated. There are several forms of dredges similar to the above, but differing from them in details.

For a description of the Eads sand-pump, see § 448.

413. In one case in France, the soil was excavated by the aid of compressed air. An 8-inch iron tube rested on the bottom, with its top projecting horizontally above the water; and compressed air was discharged through a small pipe into the lower end of the 8-inch tube. The weight of the air and water in the tube was less than an equal height of the water outside; and hence the water in the tube was projected from the top, and carried with it a portion of the mud, sand, etc. Pebbles and stones of considerable size were thus thrown out. See § 447.

414. Noted Examples.—Poughkeepsie Bridge. The Poughkeepsie bridge, which crosses the Hudson at a point about 75 miles above New York City, is founded upon cribs, and is the boldest example of timber foundation on record. It is remarkable both for the size of the cribs and for the depth of the foundation.

There are four river piers. The crib for the largest is 100 feet long, 60 feet wide at the bottom and 40 feet at the top, and 104 feet high. It is divided, by one longitudinal and six transverse walls, into fourteen compartments through which the dredge worked. The side and division walls terminate at the bottom with a 12″ × 12″ oak stick, which served as a cutting edge. The exterior walls

and the longitudinal division wall were built solid, of triangular cross section, for 20 feet above the cutting edge, and above that they were hollow. The gravel used to sink the crib was deposited in these hollow walls. The longitudinal walls were securely tied to each other by the end and cross division walls, and each course of timber was fastened to the one below by 450 1-inch drift-bolts 30 inches long. The timber was hemlock, 12 inches square. The fourteen compartments in which the clam-shell dredges worked were 10 × 12 feet in the clear. The cribs were kept level while sinking by excavating from first one and then the other of the compartments. Gravel was added to the pockets as the crib sunk. When hard bottom was reached, the dredging pockets were filled with concrete deposited under water from boxes holding one cubic yard each and opened at the bottom by a latch and trip-line.

After the crib was in position, the masonry was started in a floating caisson which finally rested upon the top of the crib. Sinking the crib and caisson separately is a departure from the ordinary method. Instead of using a floating caisson, it is generally considered better to construct a coffer-dam on top of the crib, in which to start the masonry. If the crib is sunk first, the stones which are thrown into the pockets to sink it are liable to be left projecting above the top of the crib and thus prevent the caisson from coming to a full and fair bearing.

The largest crib was sunk through about 53 feet of water, 20 feet of mud, 45 feet of clay and sand, and 17 feet of sand and gravel. It rests, at 134 feet below high water, upon a bed of gravel 16 feet thick overlying bed-rock. The timber work is 110 feet high, including the floor of the caisson, and extends to 14 feet below high water (7 feet below low water), at which point the masonry commences and rises 39 feet. On top of the masonry a steel tower 100 feet high is erected. The masonry in plan is 25 × 87 feet, and has nearly vertical faces. The lower chord of the channel span is 130 feet and the rail is 212 feet above high water.

The other piers are nearly as large as the one here described. The cribs each contain an average of 2,500,000 feet, board measure, of timber and 350 tons of wrought iron.

415. Atchafalaya Bridge. This bridge is over the Atchafalaya bayou or river, at Morgan City, La., about 80 miles west of New Orleans. The soil is alluvial to an unknown depth, and is subject

to rapid and extensive scour; and no stone suitable for piers could be found within reasonable distance. Hence iron cylinders were adopted. They are foundation and pier combined. The cylinders were sunk 120 feet below high water—from 70 to 115 feet below the mud line—by dredging the material from the inside with a Milroy excavator. Fig. 62 shows the excavator and the appliances for handling the cylinders.

FIG. 62.—SINKING IRON PILE BY DREDGING.

The cylinders are 8 feet in outside diameter. Below the level of the river bed, they are made of cast iron $1\frac{1}{4}$ inches thick, in lengths of $10\frac{1}{2}$ feet; the sections were bolted together through inside flanges with 1-inch bolts spaced 5 inches apart. Above the river bottom, the cylinders are made of wrought-iron plates $\frac{5}{8}$ inches thick, riveted together to form short cylindrical sections with angle-iron flanges. The bolts and spacing to unite the sections are the same as in the cast-iron portions.

The cylinders were filled with concrete and capped with a heavy

cast-iron plate. Two such cylinders, braced together, form the pier between two 250-feet spans of a railroad bridge.

The only objection to such piers relates to their stability. These have stood satisfactorily since 1870.

416. Hawkesbury Bridge. The bridge over the Hawkesbury River in south-eastern Australia is remarkable for the depth of the foundation. It is founded upon elliptical iron caissons 48 × 20 feet at the cutting edge, which rest upon a bed of hard gravel 126 feet below the river bed, 185 feet below high water, and 227 feet below the track on the bridge. The soil penetrated was mud and sand. The caissons were sunk by dredging through three tubes, 8 feet in diameter, terminating in bell-mouthed extensions, which met the cutting edge. The spaces between the dredging tubes and the outer shell were filled with gravel as the sinking progressed. The caissons were filled to low water with concrete, and above, with cut-stone masonry.

417. Brick Cylinders. In Germany a brick cylinder was sunk 256 feet for a coal shaft. A cylinder 25¼ feet in diameter was sunk 76 feet through sand and gravel, when the frictional resistance became so great that it could be sunk no further. An interior cylinder, 15 feet in diameter, was then started in the bottom of the larger one, and sunk 180 feet further through running quicksand. The soil was removed without exhausting the water.

A brick cylinder—outer diameter 46 feet, thickness of wall 3 feet—was sunk 40 feet in dry sand and gravel without any difficulty. It was built 18 feet high (on a wooden curb 21 inches thick), and weighed 300 tons before the sinking was begun. The interior earth was excavated slowly, so that the sinking was about 1 foot per day,—the walls being built up as it sank.

In Europe and India masonry bridge piers are sometimes sunk by this process, a sufficient number of vertical openings being left through which the material is brought up. It is generally a tedious and slow operation. To lessen the friction a ring of masonry is sometimes built inside of a thin iron shell. The last was the method employed in putting down the foundations for the new Tay bridge.*

418. Frictional Resistance. The friction between cylinders and the soil depends upon the nature of the soil, the depth sunk, and the method used in sinking. If the cylinder is sunk by either

* For an illustrated account, see *Engineering News*, vol. xiv. pp. 66–68.

of the pneumatic processes (§§ 425 and 426), the flow of the water or the air along the sides of the tube greatly diminishes the friction. It is impossible to give any very definite data.

The following table * gives the values of the co-efficient of friction † for materials and surfaces which occur in sinking foundations for bridge piers. Each result is the average of at least ten experiments. "All materials were rounded off at their face to sledge shape and drawn lengthwise and horizontally over the gravel or sand, the latter being leveled and bedded as solid as it is likely to be in its natural position. The riveted sheet iron contained twenty-five rivets on a surface of $2.53 \times 1.67 = 4.22$ square feet; the rivet-heads were half-round and $\frac{11}{16}$ inch in diameter." Notice that for dry materials and also for wet gravel and sand, the frictional resistance at starting is smaller than during motion, which is contrary to the ordinary statement of the laws of friction.

TABLE 30.

CO-EFFICIENT OF FRICTION OF MATERIALS AND SURFACES USED IN FOUNDATIONS.

KIND OF MATERIALS.	FOR DRY MATERIALS.		FOR WET MATERIALS.	
	At Beginning of Motion.	During Motion.	At Beginning of Motion.	During Motion.
Sheet iron without rivets on gravel and sand.....	0.40	0.46	0.33	0.44
" " with " " " " "	0.40	0.49	0.47	0.55
Cast iron (unplaned) on gravel and sand.........	0.37	0.47	0.36	0.50
Granite (roughly worked) on gravel and sand....	0.43	0.54	0.41	0.48
Pine (sawed) on gravel and sand................	0.41	0.51	0.41	0.50
Sheet iron without rivets on sand..............	0.54	0.63	0.37	0.32
" " with " " "	0.73	0.84	0.52	0.50
Cast iron (unplaned) on sand....................	0.56	0.61	0.47	0.38
Granite (roughly worked) on sand................	0.65	0.70	0.47	0.53
Pine (sawed) on sand...........................	0.66	0.73	0.58	0.48

419. Values from Actual Practice. *Cast Iron.* During the construction of the bridge over the Seine at Orival, a cast-iron

* By A. Schmoll in "Zeitschrift des Vereines Deutscher Ingenieure," as republished in Selected Abstracts of Inst. of C. E., vol. lii. pp. 298–302.

† The co-efficient of friction is equal to the total friction *divided by* the total normal pressure; that is to say, it is the friction per unit of pressure perpendicular to the surfaces in contact.

cylinder, standing in an extensive and rather uniform bed of gravel, and having ceased to move for thirty-two hours, gave a frictional resistance of nearly 200 lbs. per sq. ft.* At a bridge over the Danube near Stadlau, a cylinder sunk 18.75 feet into the soil (the lower 3.75 feet being "solid clay") gave a frictional resistance of 100 lbs. per sq. ft.* According to some European experiments, the friction of cast-iron cylinders in sand and river mud was from 400 to 600 lbs. per sq. ft. for small depths, and 800 to 1,000 for depths from 20 to 30 feet.† At the first Harlem River bridge, New York City, the frictional resistance of a cast-iron pile, while the soil around it was still loose, was 528 lbs. per. sq. ft. of surface; and later 716 lbs. per sq. ft. did not move it. From these two experiments, McAlpine, the engineer in charge, concluded that "1,000 lbs. per sq. ft. is a safe value for moderately fine material."‡ At the Omaha bridge, a cast-iron pile sunk 27 feet in sand, with 15 feet of sand on the inside, could not be withdrawn with a pressure equivalent to 154 lbs. per sq. ft. of surface in contact with the soil; and after removal of the sand from the inside, it moved with 210 lbs. per sq. ft.§

Wrought Iron. A wrought-iron pile, penetrating 19 feet into coarse sand at the bottom of a river, gave 280 lbs. per sq. ft.; another, in gravel, gave 300 to 335 lbs. per sq. ft.‖

Masonry. In the silt on the Clyde, the friction on brick and concrete cylinders was about 3½ tons per sq. ft.¶ The friction on the brick piers of the Dufferin (India) Bridge, through clay, was 900 lbs. per sq. ft.**

Pneumatic Caissons. For data on the frictional resistance of pneumatic caissons, see § 455.

Piles. For data on the frictional resistance of ordinary piles, see §§ 370–71.

420. COST. It is difficult to obtain data under this head, since but comparatively few foundations have been put down by this process. Furthermore, since the cost varies so much with

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* Van Nostrand's Engin'g Mag., vol. xx. pp. 121–22.

† Proc. Inst. of C. E., vol. l. p. 131.

‡ McAlpine in Jour. Frank. Inst., vol. lv. p. 105; also Proc. Inst. of C. E., vol. xxvii. p. 286.

§ Van Nostrand's Engin'g Mag., vol. viii. p. 471.

‖ Proc. Inst. of C. E., vol. xv. p. 290.

¶ *Ibid.*, vol. xxxiv. p. 35.

** *Engineering News*, vol. xix. p. 160.

the depth of water, strength of current, kind of bottom, danger of floods, requirements of navigation, etc., etc., no such data are valuable unless accompanied by endless details.

Cribs. The materials in the cribs will cost, in place, about as follows: timber from $30 to $40 per thousand feet, board measure; drift and screw bolts from 3½ to 5 cents per pound; concrete from $4 to $6 per cubic yard. Under ordinarily favorable conditions, the sinking by dredging will cost about $1 per cubic yard.

Iron Tubes. Wrought-iron plate work will cost, exclusive of freight, from 3 to 4½ cents per pound; cast-iron tubes, exclusive of freight, 1½ to 2 cents per pound.

421. For the relative cost of different methods, see Art. 6 of this chapter.

422. Conclusion. A serious objection to this method of sinking foundations is the possibility of meeting wrecks, logs, or other obstructions, in the underlying materials; but unless the freezing process (see Art. 5 of this chapter) shall prove all that is claimed for it, the method by dredging through tubes or wells is the only one that can be applied to depths which much exceed 100 feet—the limit of the pneumatic process.

Art. 4. Pneumatic Process.

424. The principle involved is the utilization of the difference between the pressure of the air inside and outside of an air-tight chamber. The air-tight chamber may be either an iron cylinder, which becomes at once foundation and pier, or a box—open below and air-tight elsewhere—upon the top of which the masonry pier rests. The former is called a *pneumatic pile;* the latter a *pneumatic caisson.* The pneumatic pile is seldom used now. There are two processes of utilizing this difference of pressure,—the *vacuum* and the *plenum.*

425. Vacuum Process. The vacuum process consists in exhausting the air from a cylinder, and using the pressure of the atmosphere upon the top of the cylinder to force it down. Exhausting the air allows the water to flow past the lower edge into the air-chamber, thus loosening the soil and causing the cylinder to sink. By letting the air in, the water subsides, after which the exhaustion may be repeated and the pile sunk still farther. The vacuum

should be obtained suddenly, so that the pressure of the atmosphere shall have the effect of a blow; hence, the pile is connected by a large flexible tube with a large air-chamber—usually mounted upon a boat,—from which the air is exhausted. When communication is opened between the pile and the receiver, the air rushes from the former into the latter to establish equilibrium, and the external pressure causes the pile to sink.

To increase the rapidity of sinking, the cylinders may be forced down by a lever or by an extra load applied for that purpose. In case the resistance to sinking is very great, the material may be removed from the inside by a sand-pump (§ 448), or a Milroy or clam-shell dredge (§ 412); but ordinarily no earth is removed from the inside. Cylinders have been sunk by this method 5 or 6 feet by a single exhaustion, and 34 feet in 6 hours.

The vacuum process has been superseded by the plenum process.

426. Plenum, or Compressed-air, Process. The plenum, or compressed-air, process consists in pumping air into the air-chamber, so as to exclude the water, and forcing the pile or caisson down by a load placed upon it. An air-lock (§ 431) is so arranged that the workmen can pass into the caisson to remove the soil, logs, and bowlders, and to watch the progress of the sinking, without releasing the pressure. The vacuum process is applicable only in mud or sand; but the compressed-air process can be applied in all kinds of soil.

427. History of Pneumatic Processes. It is said that Papin, the eminent physicist—born at Blois in 1647,—conceived the idea of employing a continued supply of compressed air to enable workmen to build under a large diving-bell. In 1779, Coulomb presented to the Paris Academy of Science a paper detailing a plan for executing all sorts of operations under water by the use of compressed air. His proposed apparatus was somewhat like that now in general use.

In England in 1831, Earl Dundonald, then Lord Cochrane, took out a patent for a device for sinking tubular shafts through earth and water, by means of compressed air. His air-lock was much like modern ones, and was to be placed at the top of the main shaft. His invention was made with a view to its use in tunneling under the Thames, and in similar enterprises. In 1841, Bush also took out a patent in England for a plan of sinking foundations by the

aid of compressed air. A German, by name G. Pfaun Muller, made a somewhat similar design for a bridge at Mayence, in 1850; but as his plan was not executed, it was, like the patents of Cochrane and Bush, little known till legal controversies in regard to patent-rights dragged them from obscurity.

428. The first practical application of the plenum process was made in France in 1841 by M. Triger. In order to reach a vein of coal on a sandy island in the Loire, opposite to Chalons, he sunk an iron tube about 40 inches in diameter, some 60 feet, by the blows of heavy weights. The fine sand was removed from the interior by means of a scoop bucket. On reaching a layer of coarse gravel, he could not force the tube through. He therefore capped his tube with an air-lock, and by compressed air forced out the water which had all the while filled the tube, and sent workmen to the bottom. The pressure he used was never greater than two atmospheres. The water was discharged through a small tube, into which, several feet from the bottom, a jet of air was allowed to enter, thus diminishing the specific gravity of the column till it was rapidly blown out. In 1845, Triger read a paper on the sinking of a tube about 6 feet in diameter to a depth of 82 feet by the same method, and suggested the use of it for the construction of deep foundations for bridges.

Dr. Potts, of England, generally has the credit of inventing the vacuum process, for which he took out a patent in 1848. Many times in sinking foundations by the vacuum process, the compressed-air process was resorted to so that men could enter the pile to remove obstructions; and finally the many advantages of the compressed-air process caused it to entirely supersede the vacuum process. At present the term "pneumatic process" is practically synonymous with compressed-air process.

429. The first foundations sunk entirely by the compressed-air process were the pneumatic piles for the bridge at Rochester, England, put down in 1851. The depth reached was 61 feet.

The first pneumatic caisson was employed at Kehl, on the eastern border of France, for the foundations of a railroad bridge across the Rhine.

430. The first three pneumatic pile foundations in America were constructed in South Carolina between 1856 and 1860. Immediately after the civil war, a number of pneumatic piles were

sunk in western rivers for bridge piers. The first pneumatic caissons in America were those for the St. Louis bridge (§ 457), put down in 1870. At that time these were the largest caissons ever constructed, and the depth reached—109 feet 8¼ inches—has not yet been exceeded.

Of late years, the pneumatic caisson has almost entirely superseded the pneumatic pile; in fact the plenum-pneumatic caisson has almost entirely superseded, except in very shallow water or in water over about 80 or 100 ft. deep, all other methods of founding bridge piers.

431. Pneumatic Piles. Although pneumatic cylinders are now rarely employed, they will be briefly described because of their historic interest.

The cylinders are made of either wrought or cast iron. The wrought-iron cylinders are composed of plates, about half an inch thick, riveted together and strengthened by angle irons on the inside, and re-inforced at the cuttting edge by plates on the outside both to increase the stiffness and to make the hole a little larger so as to diminish friction. The cast-iron cylinders are composed of sections, from 6 to 10 feet long and 2 to 8 feet in diameter, bolted together by inside flanges, the lower section being cast with a sharp edge to facilitate penetration. Two of these tubes, braced together, are employed for ordinary bridge piers; and six small ones around a large one for a pivot pier. They are filled with concrete, with a few courses of masonry or a heavy iron cap at the top.

Fig. 63 shows the arrangement of the essential parts of a pneumatic pile. The apparatus as shown is arranged for sinking by the plenum process; for the vacuum process the arrangement differs only in a few obvious particulars. The upper section constitutes the *air-lock*. The doors *a* and *b* both open downwards. To enter the cylinder, the workmen pass into the air-lock, and close the door *a*. Opening the cock *d* allows the compressed air to enter the lock; and when the pressure is equal on both sides, the door *b* is opened and the workmen pass down the cylinder by means of a ladder. To save loss of air, the air-lock should be opened very seldom, or made very small if required to be opened often.

The air-supply pipe connects with a reservoir of compressed air on a barge. If the air were pumped directly into the pile without the intervention of a storage reservoir, as was done in the early ap-

plications of the plenum process, even a momentary stoppage of the engine would endanger the lives of the workmen.

432. The soil may be excavated by ordinary hand tools, elevated to the air-lock by a windlass and bucket, and passed out through the main air-lock. Sometimes a double air-lock with one large and one small compartment is used, the former being opened only to let gangs of workmen pass and the latter to allow the passage of the

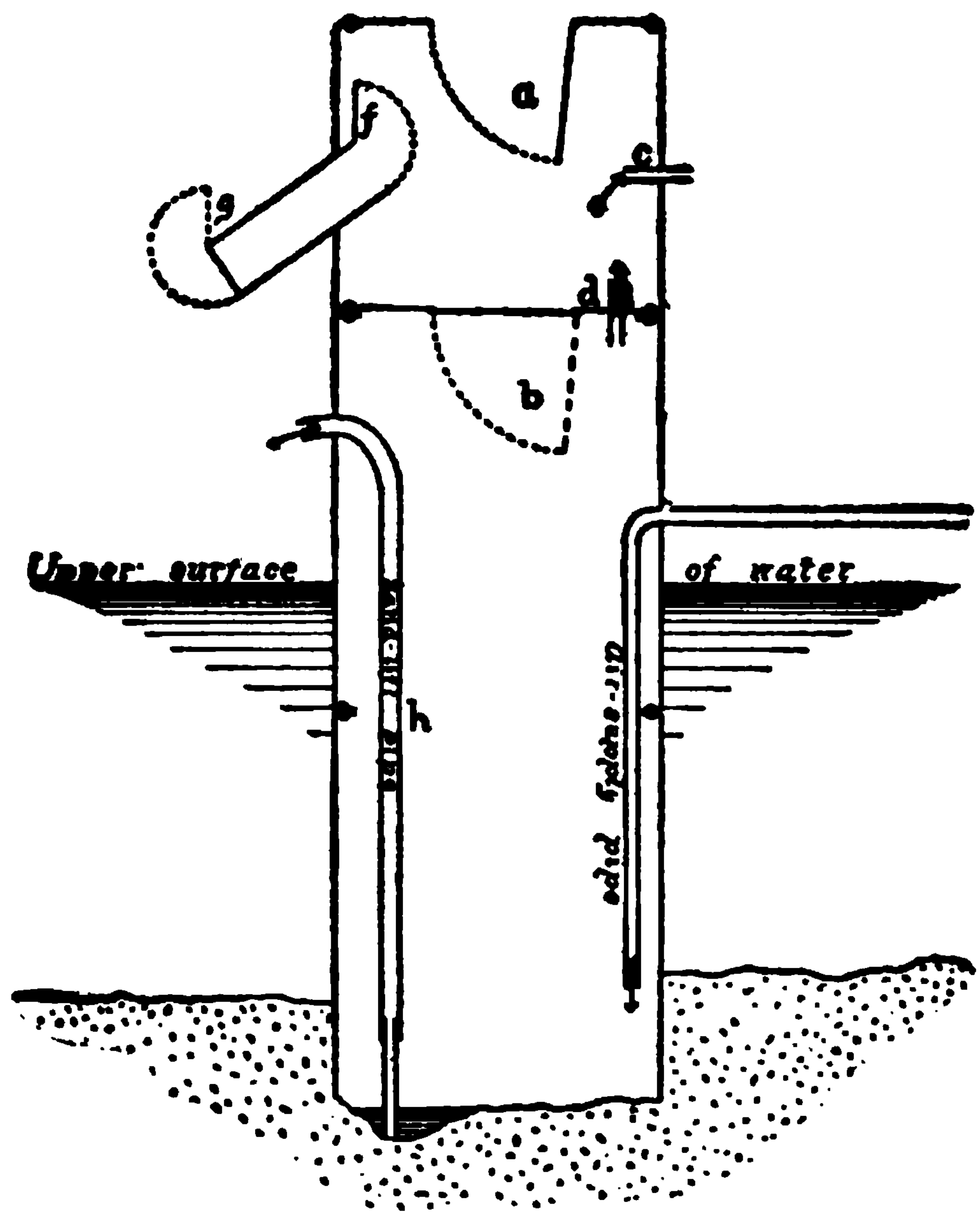

FIG. 68.—PNEUMATIC PILE.

skip, or bucket, containing the excavated material. Sometimes an auxiliary lock, *g f*, is employed. The doors *f* and *g* are so connected by parallel bars (not shown) that only one can be opened at a time. The excavated material is thrown into the chute, the door *f* is closed, which opens *g*, and the material discharges itself on the outside.

Mud and sand are blown out with the sand-lift (§ 447) or sand-pump (§ 448) without the use of any air-lock.

433. The cylinders are guided in their descent by a frame-work resting upon piles or upon two barges. One of the chief difficulties in

sinking pneumatic piles is to keep them vertical. If the cylinder becomes inclined, it can generally be righted (1) by placing wooden wedges under the lower side of the cutting edge, or (2) by excavating under the upper side so that the air may escape and loosen the material on that side, or (3) by drilling holes through the uppermost side of the cylinder through which air may escape and loosen the soil, or (4) by straining the top over with props or tackle. If several pneumatic piles are to form a pier, they should be sunk one at a time, for when sunk at the same time they are liable to run together.

434. Bearing Power. The frictional resistance of iron cylinders has been discussed in §§ 418–19, page 275–77, which see.

McAlpine, in sinking the piers of the Harlem bridge, New York City, devised a very valuable but simple and cheap method of increasing the bearing power of a pneumatic cylinder (see Fig. 64). He attached to the lower end of the cylindrical column a hollow conical iron section, the large end of which is much larger than the main cylinder. The base of the pier was still further increased by driving short sheet piles obliquely under the lower edge of the conical base and removing the soil from under them, after which the whole was filled in with concrete.*

FIG. 64.

In cold climates the contraction of the iron cylinder upon the masonry filling might rupture the former; hence, it is sometimes recommended to fill the pile below the frost line with asphaltic concrete. It has also been proposed to line the cylinders with thick, soft wood staves, which will compress under the contraction of the iron cylinder. However, the danger from this cause is not very serious; for, after the concrete has set, it is strong enough to support the load if the iron case were removed.

435. After the cylinder has reached the required depth, concrete enough to seal it is laid in compressed air; and when this has set, the remainder can be laid in the open air. A short distance at the top is usually filled with good masonry, and a heavy iron cap put over all.

* Jour. Frank. Inst., vol. lv. pp. 96 and 177.

436. PNEUMATIC CAISSONS. A pneumatic caisson is an immense box—open below, but air-tight and water-tight elsewhere,—upon the top of which the masonry pier is built. The essential difference between the pneumatic pile and the pneumatic caisson is one of degree rather than one of quality. Sometimes the caisson envelops the entire masonry of the pier; but in the usual form the masonry envelops the iron cylinder and rests upon an enlargement of the lower end of it. The pneumatic pile is sunk to the final depth before being filled with concrete or masonry; but with the caisson the masonry is built upward while the whole pier is being sunk downward, the masonry thus forming the load which forces the caisson into the soil. A pneumatic caisson is, practically, a gigantic diving bell upon the top of which the masonry of the pier rests.

Fig. 65 is a section of a pier of the bridge across the Missouri River near Blair, Neb.,* and shows the general arrangement of the pier and pneumatic caisson. The tube extending through the middle of the caisson and pier, known as the *air-shaft*, is for the ascent and descent of the men. The *air-lock*—situated at the junction of the two cylinders which form the air-shaft—consists of a short section of a large cylinder which envelops the ends of the two sections of the air-shaft, both of which communicate with the air-lock by doors as shown in Fig. 65. The apartment in which the men are at work is known as the *working chamber* or *air-chamber*. The small cylinders shown on each side of the air-shaft are employed in supplying concrete for filling the working chamber when the sinking is completed. The pipes seen in the air-chamber and projecting above the masonry are employed in discharging the mud and sand, as will be described presently. The timbers which appear in the lower central portion of the working chamber are parts of the trusses which support the central portions of the roof of the caisson.

The masonry is usually begun about 2 feet below low water, the space intermediate between the masonry and the roof of the working chamber being occupied by timber crib-work, either built solid or filled with concrete. In Fig. 65 the masonry rests directly upon the roof of the air-chamber, which construction was adopted for the channel piers of this bridge to reduce to a minimum the obstruction to the flow of the water.

Frequently a coffer-dam is built upon the top of the crib (see

* From the report of Geo. S. Morison, chief engineer of the bridge.

Plate I); but in this particular case the masonry was kept above the surface of the water, hence no coffer-dam was employed. When

FIG. 65.—PNEUMATIC CAISSON.—BLAIR BRIDGE.

the coffer-dam is not used, it is necessary to regulate the rate of sinking by the speed with which the masonry can be built, which is liable to cause inconvenience and delay. When the coffer-dam is

dispensed with, it is necessary to go on with the construction of the masonry whether or not the additional weight is needed in sinking the caisson.

437. The details of the construction of pneumatic caissons can be explained best by the description of a particular case.

438. Foundation of the Havre de Grace Bridge. Folding Plate I * shows the details of the construction of the caisson, crib, and coffer-dam employed in 1884 in sinking pier No. 8 of the Baltimore and Ohio R. R. bridge across the Susquehanna River at Havre de Grace, Md. The timber work of Fig. 66 (page 293) also shows some of the details of the construction of the walls of the working chamber.

439. The Caisson. The details of the construction of the caisson are as follows: Six courses of timber, 12 × 12 inch, one lying on top of the other, formed the skeleton of the walls of the working chamber. These timbers were first put up with a batter of ⅝ of an inch horizontal to 1 foot vertical; they were not halved at the corners, but every alternate piece was carried through with a full section, "log-house" fashion. These timbers were fastened at the corners, intersections, and several intermediate points, with drift-bolts (§ 381) 1 inch square and 22 inches long. Inside of this timber shell, three courses of 3-inch plank, placed diagonally, were spiked to the horizontal timbers and to each other by 6-inch and 7-inch boat-spikes. Inside of the diagonal planking was another course of 3-inch plank placed vertically and well spiked, the head of each spike being wrapped with oakum to prevent leakage. The vertical seams were thoroughly calked.

A strong and thoroughly braced truss (see also Fig. 66, page 293) was next erected longitudinally through the center of the working chamber. The first course in the deck of the working chamber was then placed in position on the central truss and side walls. The working chamber was 9 feet 3 inches high from bottom of shoe to the underside of deck, which was higher than required for working, but was adopted so as to permit greater depth of the central truss. Outside of the horizontal timbers, after they had been adzed to a true surface, were then placed the 12- by 14-inch sticks (shown at the ex-

* Compiled from the original working drawings. The accompanying description is from personal inspection aided by an article in *Engineering News* by Col. Wm. M. Patton, engineer in charge.

treme left of Fig. 66) 15 feet long, extending 2 feet below the bottom horizontal timber and having their lower ends beveled as shown. These timbers extended 6 feet above the horizontal members, and were shouldered at the upper end so that three of the deck courses rested upon them. Four screw-bolts were passed through each outside post and through the entire wall; and, in addition to these, two drift-bolts, 1 inch square and 30 inches long, in each vertical served to more thoroughly bind the wall together. This compound of timber and planking formed the walls of the working chamber. After the first deck course was in place, a few pieces of the second course were laid diagonally to give it stiffness; the underside of this deck or roof was then lined with planks and thoroughly calked, and a false bottom put into the working chamber preparatory to launching it.

After the caisson was launched the deck courses, eight in all, were put on. The first course was made of single-length timbers, reaching from inside to inside of the vertical wall posts, and resting on top of the horizontal timbers and inside planking and also on the top chord of the central truss, and being fastened to these members by 22-inch drift-bolts. The second course was laid diagonally and was made of varying lengths of timbers. The third course was laid from side to side across the caisson, and the fourth course longitudinally and resting on the shoulders of the 12 × 14 inch verticals. The fifth course was laid across, the sixth diagonally—crossing the second course,—and the seventh and eighth courses extended to the extreme outside limits of the caisson and rested on the heads of the vertical posts. This general arrangement of the top courses, resting as they did on the heads and shoulders of the outside verticals, gave a direct bearing on the posts and relieved the wall bolts of the great shearing strain to which they would otherwise have been subjected.

The outside posts were bolted to the deck courses by one 3-foot screw-bolt and two 30-inch drift-bolts, fastening them to the longitudinal and diagonal courses respectively. The several deck courses were bolted to each other by 22-inch drift-bolts (not shown in the illustrations), spaced 5 feet apart along each stick. All the timbers in the deck were bedded in cement mortar and the vertical joints were grouted, so as to give a full and uniform bearing for each stick and also decrease the leakage and danger from fire.

The center truss (see also Fig. 66) was constructed to bear a uni-

formly distributed load, or to act as a cantilever. It was composed of a top and bottom chord, each made of two 12 × 12 inch sticks, with posts and diagonals of wood, and vertical and diagonal tie-rods $1\frac{1}{2}$ inch in diameter; the iron vertical rods extended through the first deck courses, and the top chord was also bolted to the deck with drift-bolts. The object of this was to enable the truss to act as a stiffening rib to the deck, independently of its action as a girder. The bottom chord was also extended to the ends, and by means of straps and bolts acted both as a strut and tie-brace for the ends of the caisson, and constituted the only end bracing.

The sides of the caissons were braced against outside pressures by 16×16 inch timbers abutting against the walls and bottom chord of the center truss, and against pressure from the inside by 2-inch iron tie-rods extending from out to out of the caisson, none of which are shown. All the timber used, except the planking and outside posts and the bracing in the working chamber, was 12×12 inch. Iron straps, extending 6 feet on the sides and ends, were placed at the corners and bolted to the caisson timbers. These straps were made of bar-iron 3×1 inch and prevented spreading of the walls of the caisson under excessive pressure within. Planks were spiked to the lower part of the posts; and also a narrow plank, called a shoe, was spiked under the bottom of the posts (see Fig. 66).

440. "The construction was simple and strong; in no case was there any bending or springing of the walls. The arrangement of the cutting edge with square shoulders was a departure from the ordinary V-shape (compare Figs. 65 and 66, pages 285 and 293), and was found to possess many advantages. It enabled the men to better regulate the sinking of the caisson by giving an increased bearing surface. With this support, the material could be cleaned out from under one side or end; the caisson could be leveled; and, if the material was softer in one spot than another, the caisson could be prevented from tipping. It further afforded a good surface for blocking up when it was found desirable to support the caisson during the removal of the material; and it gave also greater security in case of a 'blow-out' or the failure of air-pressure."*

When it is anticipated that gravel or bowlders will be met with in sinking, the cutting edge is usually shod with iron. The iron cutting edge was omitted in all the caissons for this bridge, and it is

* Col. Wm. M. Patton, engineer in charge for the railroad company.

claimed that the experience here shows that "in no case is an iron shoe either advantageous or necessary."

441. The Crib. The construction of the crib is shown very fully in Plate I. The timbers were all 12 × 12 inches square, bolted to each other by 22-inch drift-bolts—spaced 5 or 6 feet apart,—and were dovetailed at the corners and connections. The parts of all the walls of the crib were firmly bolted to the deck of the caisson.

Ordinarily the division walls of the crib are built vertically from top to bottom; but in this case, they were off-set, as shown, to secure a better bond in the mass of concrete. If the walls are built solid from top to bottom, the concrete filling is thereby divided into a number of separate monolithic columns; but in the construction as above, the concrete forms practically a single solid mass. The walls are built solid, owing to the difficulty of getting the concrete thoroughly packed in around so many timbers. Large stones, such as could be handled by one man, were bedded in mortar as the successive layers of concrete were formed, and over and around these another layer of concrete was rammed. In most localities there is but little difference in cost between a solid timber crib and one with timber pockets filled with concrete.

442. The Coffer-dam. Uprights were first placed at intervals of about 5½ feet, and connected by mortise and tenon to caps and sills. This frame-work was held down to the crib by rods 2 inches in diameter, having hooks at the lower end which passed into eye-bolts in the sides of the crib. On the sides of the dam, the upper end of these rods passed through 12 × 12 inch timbers resting on the sides of the dam and projecting about 2 feet outside; and at the ends of the dam, they passed through short pieces bolted to one of the cross timbers and projecting beyond the end of the dam.

Owing to the great depth required, the coffer-dam was built in sections, the connecting rods being made in sections with swivel connections. The second section was not added until the depth sunk required it. When the top section of the dam was put on, the projecting ends of the timbers through which the connecting rods passed were sawed off. The bottom section was sheeted with three courses of 3-inch plank, and the top section with two thicknesses. The joint between the coffer-dam and the crib, and also the sheeting, were well calked.

The sides of the coffer-dam were braced against the pressure of

the water, by 12 × 12 inch timbers resting on the top of each section, and by a system of bracing in the middle of each section. When the masonry was completed, the coffer-dam was removed by disconnecting the vertical rods.

443. Machinery Barge. The machinery barge was an ordinary flat-boat fitted up for the purpose. At one end of the barge there were three boilers each of fifty horse-power. In the middle were two large air-compressors, designed by the contracting engineer, Gen. Wm. Sooy Smith. One furnished all the compressed air required, the other being ready for use in case of any accident or break-down. At the other end of the boat were two Worthington steam pumps to furnish water for the excavating plant used in the caisson. There were also a small engine and a dynamo which furnished the current for the electric lamps used in the caisson and, at night, on the outside.

444. Material Required. Table 31 gives the dimensions and quantities of materials in the pneumatic foundations of this bridge, and Table 32 (page 302) gives the cost.

TABLE 31.

DIMENSIONS AND QUANTITIES OF MATERIALS IN FOUNDATIONS OF HAVRE DE GRACE BRIDGE.*

DESCRIPTION.	NUMBER OF THE PIER.				
	II.	III.	IV.	VIII.	IX.
DIMENSIONS:					
Caissons:—length at bottom, in feet..........	63.8	67.8	79.4	70.9	78.2
width " " " "	25.9	25.9	32.8	32.6	42.3
height from cutting edge, in feet..	17.2	17.2	17.2	17.2	19.3
height of working chamber, in feet	9.2	9.2	9.2	9.2	9.2
Crib:—length, in feet..........................	61.5	61.5	77.6	69.1	76.4
width, " "	24.2	24.2	31.1	30.8	40.5
height, " "	40.0	42.0	22.2	41.0	32.8
QUANTITIES:					
Timber in the caisson, feet, board measure.	203,473	215,565	316,689	281,540	465,125
" " " crib, " " "	179,989	197,910	143,998	219,680	306,894
" " " coffer-dam." " "	2,068	31,517	103,518	85,759	126,532
Concrete in working chamber, cubic yards...	330	401	631	559	839
" " crib, shafts, etc., " " ...	1,649	1,893	1,685	2,581	3,172
" below cutting edge, " " ...	000	623	126	526	624
Iron, screw-bolts, pounds....................	11,318	15,651	32,881	31,026	33,435
drift-bolts, "	34,181	36,832	40,909	44,861	59,245
spikes, "	4,638	700	11,730	10,039	11,237
cast washers, "	2,472	2,572	3,392	3,235	3,535

* The data by courtesy of Sooysmith & Co., contractors for the pneumatic foundations.

445. POSITION OF THE AIR-LOCK. Before the construction of the St. Louis bridge, the air-lock had always been placed at the top

of the air-shaft, and was of such construction that to lengthen the shaft, as the caisson sunk, it was necessary to detach the lock, add a section to the shaft, and then replace the lock on top. This was not only inconvenient and an interruption to the other work, but required the men to climb the entire distance under compressed air, which is exceedingly fatiguing (see § 460). To overcome these objections, Eads placed the air-lock at the bottom of the shaft. This position is objectionable, since in case of a "blow-out," *i. e.*, a rapid leakage of air,—not an unfrequent occurrence,—the men may not be able to get into the lock in time to escape drowning. If the lock is at the top, they can get out of the way of the water by climbing up in the shaft.

At the Havre de Grace bridge, the air-shaft was constructed of wrought iron, in sections 15 feet long. The air-lock was made by placing diaphragms on the inside flanges of the opposite ends of the top section. A new section and a third diaphragm could be added without disturbing the air-lock; and when the third diaphragm was in place, the lower one was removed preparatory to using it again. Some engineers compromise between these two positions, and leave the air-lock permanently at some intermediate point in the pier (see Fig. 65, page 285).

446. Excavators. In the early application of the pneumatic method, the material was excavated with shovel and pick, elevated in buckets or bags by a windlass, and stored in the air-lock. When the air-lock was full, the lower door was closed, and the air in the lock was allowed to escape until the upper door could be opened, and then the material was thrown out. This method was expensive and slow.

In the first application of the pneumatic process in America (§ 430), Gen. Wm. Sooy Smith invented the auxiliary air-lock, *g f*, Fig. 63 (page 282), through which to let out the excavated material. The doors, *f* and *g*, are so connected together that only one of them can be opened at a time. The excavated material being thrown into the chute, the closing of the door *f* opens *g*, and the material slides out. This simple device is said to have increased threefold the amount of work that could be done.

447. Sand-lift. This is a device, first used by Gen. Wm. Sooy Smith, for forcing the sand and mud out of the caisson by means of the pressure in the working chamber. It consists of a pipe,

reaching from the working chamber to the surface (see Fig. 63 and Plate I), controlled by a valve in the working chamber. The sand is heaped up around the lower end of the pipe, the valve opened, and the pressure forces a continuous stream of air, sand, and water up and out. For another application of this principle, see § 413.

In sand, this method of excavating is very efficient, being eight to ten times as expeditious as the auxiliary air-lock. Of course, the efficiency varies with the depth, *i. e.*, with the pressure. When the soil is so impervious that the water in the working chamber can not be forced out under the edge of the caisson, it is made to pass through the sand-lift pipe.

The "goose-neck," or elbow at the top of the discharge pipe, is worn away very rapidly by the impact of the ascending sand and pebbles. At the Havre de Grace bridge, it was of chilled iron 4 inches thick on the convex side of the curve, and even then lasted only two days. At the Brooklyn bridge, the discharge pipe terminated with a straight top, and the sand was discharged against a block of granite placed in an inclined position over the upper end.

Although the sand-lift is efficient, there are some objections to it: (1) forcing the sand out by the pressure in the cylinder decreases the pressure, which causes, particularly in pneumatic piles or small caissons, the formation of vapors so thick as to prevent the workmen from seeing; (2) the diminished pressure allows the water to flow in under the cutting edge; and (3) if there is much leakage, the air-compressors are unable to supply the air fast enough.

448. Mud-pump. During the construction of the St. Louis bridge, Capt. Eads invented a mud-pump, which is free from the above objections to the sand-lift, and which in mud or silt is more efficient than it. This device is generally called a sand-pump, but is more properly a mud-pump.

The principle involved in the Eads pump is the same as that employed in the atomizer, the inspirator, and the injector; viz., the principle of the induced current. This principle is utilized by discharging a stream of water with a high velocity on the outside of a small pipe, which produces a partial vacuum in the latter; when the pressure of the air on the outside forces the mud through the small pipe and into the current of water by which the mud is carried away. The current of water is the motive power.

Fig. 66 is an interior view of the caisson of the Baltimore and Ohio R. R. bridge at Havre de Grace, Md., and shows the general arrangement of the pipes and mud-pump. The pump itself is a

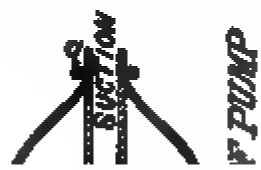

Fig. 65.

hollow pear-shaped casting, about 15 inches in diameter and 15 inches long, a section of which is shown in the corner of Fig. 66. The water is forced into the pump at *a*, impinges against the conical casing, *d*, flows around this lining and escapes upwards through a narrow annular space, *f*. The interior casing gives the water an even distribution around the end of the suction pipe. The flow of the water through the pump can be regulated by screwing the suction pipe in or out, thus closing or opening the annular space, *f*. To prevent the too rapid feeding or the entrance of lumps, which might choke the pipe, a strainer—simply a short piece of pipe, plugged at the end, having a series of ½-inch to ¾-inch holes bored in it—was put on the bottom of the suction pipe. The discharge pipe of the mud-pump terminates in a "goose-neck" through which the material is discharged horizontally.

The darkly shaded portions of the section of the pump wear away rapidly; and hence they are made of the hardest steel and constructed so as to be readily removed. Different engineers have different methods of providing for the renewal of these parts, the outline form of the pump varying with the method employed. The pump used at the St. Louis bridge was cylindrical in outline, but otherwise essentially the same as the above.

449. In order to use the mud-pump, the material to be excavated is first mixed into a thin paste by playing upon it with a jet of water. This pump is used only for removing mud, silt, and soil containing small quantities of sand; pure sand or soil containing large quantities of sand is "blown out" with the sand-lift.

The water is delivered to the mud-pump under a pressure, ordinarily, of 80 or 90 pounds to the square inch. At the St. Louis bridge, it was found that a mud-pump of 3½-inch bore was capable of raising 20 cubic yards of material 120 feet per hour, the water pressure being 150 pounds per square inch.*

450. Water-column. A combination of the pneumatic process and that of dredging in the open air through tubes has been employed extensively in Europe. It seems to have been used first at the bridge across the Rhine at Kehl. The same method was used at the Brooklyn bridge. The principle is rudely illustrated in

* History of the St. Louis Bridge, p. 213.

Fig. 67. The central shaft, which is open top and bottom, projects a little below the cutting edge, and is kept full of water, the greater height of water in the column balancing the pressure of the air in the chamber. The workmen simply push the material under the edge of a water-shaft, from whence it is excavated by a dredge (§ 412).

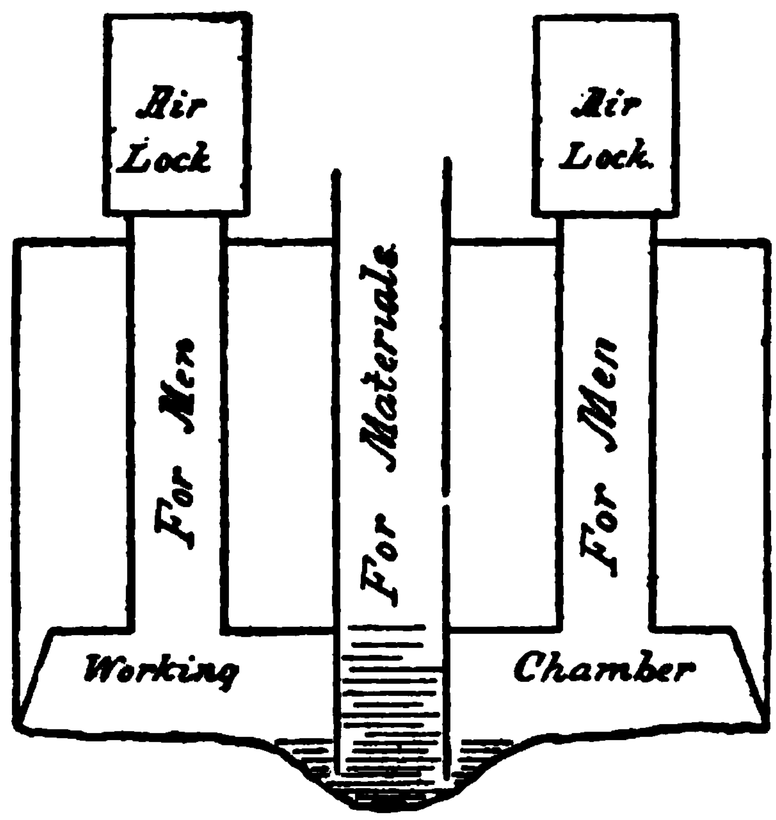

FIG. 67.

451. Blasting. Bowlders or points of rock may be blasted in compressed air without any appreciable danger of a "blow-out" or of injuring the ear-drums of the workmen. This point was settled in sinking the foundations of the Brooklyn bridge; and since then blasting has been resorted to in many cases. Bowlders are sometimes "carried down," *i. e.*, allowed to remain on the surface of the soil in the working chamber as the excavation proceeds, and subsequently imbedded in the concrete with which the air-chamber is filled.

452. RATE OF SINKING. The work in the caisson usually continues day and night, winter and summer. The rate of progress varies, of course, with the kind of soil, and particularly with the number of bowlders encountered. At the Havre de Grace bridge, the average rate of progress was 1.37 ft. per day; at Plattsmouth, 2.22 ft.; and at Blair, 1.75 ft. per day.

453. GUIDING THE CAISSON. Formerly it was the custom to control the descent of the caisson by suspension screws connected with a frame-work resting upon piles or pontoons. In a strong current or in deep water, it may be necessary to support the caisson partially in order to govern its descent; but ordinarily the suspension is needed only until the caisson is well imbedded in the soil. The caisson may be protected from the current by constructing a break-water above and producing dead water at the pier site.

After the soil has been reached, the caisson can be kept in its course by removing the soil from the cutting edge on one side or the other of the caisson. In case the caisson does not settle down

after the soil has been removed from under the cutting edge, a reduction of a few pounds in the air pressure in the working chamber is usually sufficient to produce the desired result. At the Havre de Grace bridge, it was found that by allowing the discharged material to pile up against the outside of the caisson, the latter could bemoved laterally almost at will. The top of the caisson was made 3 feet larger, all round, than the lower course of masonry, to allow for deviation in sinking. The deviation of the caisson, which was founded 90 feet below the water, was less than 18 inches, even though neither suspension screws nor guide piles were employed.

In sinking the foundations for the bridge over the Missouri River near Sibley, Mo., it was necessary to move the caisson considerably horizontally without sinking it much farther. This was accomplished by placing a number of posts—12 inches square—in an inclined position between the roof of the working chamber and a temporary timber platform resting on the ground below. When these posts had been wedged up to a firm bearing, the air pressure was released. The water flowing into the caisson loosened the soil on the outside, and the weight of the caisson coming on the inclined posts caused them to rotate about their lower ends, which forced the caisson in the desired direction. In this way, a lateral movement of 3 or 4 feet was secured while sinking about the same distance.

A caisson is also sometimes moved laterally, while sinking, by attaching a cable which is anchored off to one side and kept taut.

454. A new method of controlling the descent of the caisson has been recently introduced, which is specially valuable in swift currents or in rivers subject to sudden rises. It was used first in the construction of the piers for a bridge across the Yazoo River near Vicksburg, Miss. A group of 72 piles, each 40 feet long, was driven into the river bed, and sawed off under the water ; the caisson was then floated into place, and lowered until the heads of the piles rested against the roof of the working chamber. As the work proceeded, the piles were sawed off to allow the caisson to sink. One of the reasons for employing piles in this case, was that, if the caisson did not finally rest upon bed-rock, they would assist in supporting the pier.

That such ponderous masses can be so certainly guided in their

descent to bed-rock, is not the least valuable nor least interesting fact connected with this method of sinking foundations.

455. Frictional Resistance. At the Havre de Grace bridge, the normal frictional resistance on the timber sides of the pneumatic caisson was 280 to 350 lbs. per sq. ft. for depths of 40 to 80 feet, the soil being silt, sand, and mud; when bowlders were encountered, the resistance was greater, and when the air escaped in large quantities the resistance was less. At the bridge over the Missouri River near Blair, Neb., the frictional resistance usually ranged between 350 and 450 lbs. per sq. ft., the soil being mostly fine sand with some coarse sand and gravel and a little clay. At the Brooklyn bridge the frictional resistance at times was 600 lbs. per sq. ft. At Cairo, in sand and gravel, the normal friction was about 600 lbs. per sq. ft.

For data on the friction of iron cylinders and masonry shafts, see §§ 418–19, pages 275–77; and for data on the friction of ordinary piles, see §§ 370–72, pages 247–48.

456. Filling the Air-chamber. When the caisson has reached the required depth, the bottom is leveled off—by blasting, if necessary,—and the working chamber and shafts are filled with concrete. Sometimes only enough concrete is placed in the bottom to seal the chamber water-tight, and the remaining space is filled with sand. This was done at the east abutment of the St. Louis bridge, the sand being pumped in from the river with the sand-pump previously used for excavating the material from under the caisson.

457. Noted Examples. The St. Louis Bridge. The foundations of the steel-arch bridge over the Mississippi at St. Louis are the deepest ever sunk by the pneumatic process, and at the time of construction (1870) they were also very much the largest. The caisson of the east abutment was an irregular hexagon in plan, 83 × 70 feet at the base, and 64 × 48 feet at the top—14 feet above the cutting edge. The working chamber was 9 feet high. The cutting edge finally rested on the solid rock 94 feet below low water. The maximum emersion was 109 feet $8\frac{1}{4}$ inches, the greatest depth at which pneumatic work has yet been done. The other caissons were almost as large as the one mentioned above, but were not sunk as deep.

The caissons were constructed mainly of wood; but the side

walls and the roof were covered with plate iron to prevent leakage, and strengthened by iron girders on the inside. This was the first pneumatic caisson constructed in America; and the use of large quantities of timber was an important innovation, and has become one of the distinguishing characteristics of American practice. In all subsequent experience in this country (except as mentioned in § 458), the iron lining for the working chamber has been dispensed with. The masonry rested directly upon the roof of the caisson, *i. e.*, no crib-work was employed. In sinking the first pneumatic foundation an iron coffer-dam was built upon the top of the caisson; but the last—the largest and deepest—was sunk without a coffer-dam,—a departure from ordinary European practice, which is occasionally followed in this country (see § 436).

458. The Brooklyn Bridge. The foundations of the towers of the suspension bridge over the East River, between New York City and Brooklyn, are the largest ever sunk by the pneumatic process. The foundation of the New York tower, which was a little larger and deeper than the other, was rectangular, 172 × 102 feet at the bottom of the foundation, and 157 × 77 feet at the bottom of the masonry. The caisson proper was 31½ feet high, the roof being a solid mass of timber 22 feet thick. The working chamber was 9½ feet high. The bottom of the foundation is 78 feet below mean high tide, and the bottom of the masonry is 46½ feet below the same. From the bottom of the foundation to the top of the balustrade on the tower is 354 feet, the top of the tower being 276 feet above mean high tide.

To make the working chamber air-tight, the timbers were laid in pitch and all seams calked; and in addition, the sides and the roof were covered with plate iron. As a still further precaution, the inside of the air-chamber was coated with varnish made of rosin, menhaden oil, and Spanish brown.

For additional details see the several annual reports of the engineers in charge, and also numerous articles in the engineering newspapers and magazines from 1869 to 1872.

459. Forth Bridge. For an illustrated account of the pneumatic foundation work of the bridge across the Frith of Forth, England, see *Engineering News*, vol. xiii. pages 242–43. The caissons employed there differed from those described above (1) in being made almost wholly of iron, (2) in an elaborate system of cages for

hoisting the material from the inside, and (3) in the use of interlocked hydraulic apparatus to open and close the air-locks. Each of the two deep-water piers consists of four cylindrical caissons 70 feet in diameter the deepest of which rests 96 feet below high tide.

460. Physiological Effect of Compressed Air. In the application of the compressed-air process, the question of the ability of the human system to bear the increased pressure of the air becomes very important.

After entering the air-lock, as the pressure increases, the first sensation experienced is one of great heat. As the pressure is still further increased a pain is felt in the ear, arising from the abnormal pressure upon the ear-drum. The tubes extending from the back of the mouth to the bony cavities over which this membrane is stretched, are so very minute that compressed air can not pass through them with a rapidity sufficient to keep up the equilibrium of pressure on both sides of the drum (for which purpose the tubes were designed by nature), and the excess of pressure on the outside causes the pain. These tubes can be distended, thus relieving the pain, by the act of swallowing, or by closing the nostrils with the thumb and finger, shutting the lips tightly, and inflating the cheeks. Either action facilitates the passage of the air through these tubes, and establishes the equilibrium desired. The relief is only momentary, and the act must be repeated from time to time, as the pressure in the air-lock increases. This pain is felt only while the air in the lock is being "equalized," *i. e.*, while the air is being admitted, and is most severe the first time compressed air is encountered, a little experience generally removing all unpleasant sensations. The passage through the lock, both going in and coming out, should be slow; that is to say, the compressed air should be let in and out gradually, to give the pressure time to equalize itself throughout the various parts of the body.

When the lungs and whole system are filled thoroughly with the denser air, the general effect is rather bracing and exhilarating. The increased amount of oxygen breathed in compressed air very much accelerates the organic functions of the body, and hence labor in the caisson is more exhaustive than in the open air; and on getting outside again, a reaction with a general feeling of prostration sets in. At moderate depths, however, the laborers in the caisson,

after a little experience, feel no bad effects from the compressed air, either while at work or afterwards.

Remaining too long in the working chamber causes a form of paralysis, recently named *caisson disease*, which is sometimes fatal. The injurious effect of compressed air is much greater on men addicted to the use of intoxicating liquors than on others. Only sound, able-bodied men should be permitted to work in the caisson.

In passing through the air-lock on leaving the air-chamber, the workman experiences a great loss of heat owing (1) to the expansion of the atmosphere in the lock, (2) to the expansion of the free gases in the cavities of the body, and (3) to the liberation of the gases held in solution by the liquids of the body. Hence, on coming out the men should be protected from currents of air, should drink a cup of strong hot coffee, dress warmly, and lie down for a short time.

461. No physiological difficulty is encountered at small depths; but this method is limited to depths not much exceeding 100 feet, owing to the deleterious effect of the compressed air upon the workmen. At the east abutment of the St. Louis bridge (§ 457), the caisson was sunk 110 feet below the surface of the water. Except in this instance, the compressed-air process has never been applied at a greater depth than about 90 feet. Theoretically, the depth, in feet, of the lower edge of the caisson below the surface divided by 33 is equal to the number of atmospheres of pressure. The pressure is never more than this, and sometimes, owing to the frictional resistance to the flow of the water through the soil, it is a little less. Therefore the depth does not exactly indicate the pressure; but the rule is sufficiently exact for this purpose. At St. Louis, at a depth of 110 feet, the men were able to work in the compressed air only four hours per day in shifts of two hours each, and even then worked only part of the time they were in the air-chamber.

With reasonable care, the pneumatic process can be applied at depths less than 80 or 90 feet without serious consequences. At great depths the danger can be greatly decreased by observing the following precautions, in addition to those referred to above: (1) In hot weather cool the air before it enters the caisson;* (2) in cold

* This was done in 1888 at the bridge over the Ohio River at Cairo, Ill.—probably the first example. The temperature of the air was reduced 20° F.

weather warm the air in the lock when the men come out; and (3) raise and lower them by machinery.

For an exhaustive account of the various aspects of this subject, see Dr. Smith's article on the "Physiological Effect of Compressed Air," in he Report of the Engineer of the Brooklyn Bridge.*

462. Cost. The contract for pneumatic foundation is usually let at specified prices per unit for the materials left permanently in the structure and for the material excavated, including the necessary labor and tools. The prices for material in place are about as follows: Timber in caisson proper, from $40 to $50 per thousand feet, board measure, according to the locality in which the work is done; and the timber in the crib-work and coffer-dam about $5 to $7 per thousand less. The concrete, which is usually composed of broken stone and sufficient 1 to 2 or 1 to 3 Portland cement mortar to completely fill the voids, costs, exclusive of the cement, from $5 to $7 per cubic yard for that in the crib, and about twice this sum for that in the air-chamber and under the cutting edge. The wrought-iron spikes, drift-bolts, screw-bolts, and cast-iron washers cost from 3½ to 6 cents per pound.† The caisson and filling costs from $14 to $20 per cubic yard; and the crib and filling from $8 to $10.

The price for sinking, including labor, tools, machinery, etc., ranges, according to the kind of soil, from 18 to 40, or even 50, cents per cubic foot of the volume found by multiplying the area of the caisson at the cutting edge by the final depth of the latter below low water. In sand or silt the cost is 18 to 20 cents, and in stiff clay and bowlders 40 to 50 cents.

463. Examples. The table on page 302 gives the details of the cost of the pneumatic foundation of the Havre de Grace bridge, as fully described in §§ 438–44.

The table on page 303 gives the details of the cost of the pneumatic caissons of the bridge across the Missouri River near Blair, Neb. The caissons (Fig. 65, page 285) were 54 feet long, 24 feet wide, and 17 feet high. In the two shore piers, Nos. I and IV of the table, the caissons were surmounted by cribs 20 feet high; but in the channel piers, the masonry rested directly upon the roof of the

* Prize Essay of the Alumni Association of the College of Physicians and Surgeons of New York City, 1873.

† There are usually from 140 to 150 pounds of iron per thousand feet (board measure) of timber.

TABLE 32.

COST, TO THE R. R. CO., OF FOUNDATIONS OF HAVRE DE GRACE BRIDGE.*

ITEMS.	NUMBER OF THE PIER.				
	II.	III.	IV.	VIII.	IX.
Depth of cutting edge below low water, feet	68.8	70.7	59.9	76.0	65.0
Depth of cutting edge below mud line, feet	55.5	58.7	32.3	55.2	32.6
Displacement below low water, cu. ft.	112,124	123,402	159,588	189,578	231,691
" " mud line, cu. ft	94,504	106,269	84,014	127,586	107,836
Caisson: timber, @ $46.80 per M	$9,522.54	$10,068.44	$14,820.94	$13,176.07	$21,767.85
iron, @ 5¼ cts. per lb	1,456.12	1,587.15	2,596.23	2,242.40	3,295.38
concrete, @ $17.50 per cu. yd.	5,775.00	7,017.50	13,247.50	18,987.50	25,602.50
total cost, per net cu. yd	16.82	18.87	19.19	24.34	22.10
Crib: timber, @ $46.80 per M	8,421.14	9,262.19	6,738.87	8,936.58	9,538.96
iron, @ 5¼ cts. per lb	1,291.14	1,454.85	1,179.86	1,749.35	1,445.93
concrete, @ $8.50 per net cu. yd	14,016.50	16,090.50	13,897.50	21,943.50	26,962.00
total cost, per cu. yd	10.76	11.10	10.91	9.91	10.09
Coffer-dam: timber, @ $46.80 per M	96.78	1,375.00	5,078.64	4,018.52	5,921.70
iron, @ 5¼ cts. per lb.	14.45	236.15	892.29	684.20	899.22
Cost of sinking, @ 20 cts. per cu. ft. of displacement below low water	22,424.80	24,680.40	31,917.60	37,915.60	46,338.20
Concrete below cutting edge, @ $17.50	000	10,902.50	2,205.00	9,205.00	10,920.00
Total cost of foundation	63,018.47	71,792.18	90,368.93	109,648.72	141,772.44
Total cost per cu. yd. of foundation below masonry, including coffer-dams.	19.93	21.58	25.20	23.30	23.44

Average total cost of the foundation, to R. R. Co., per net cubic yard........$22.69.

caisson. The work was done, in 1882–83, by the bridge company's men under the direction of the engineer.

464. In 1869–72, thirteen cylinders were sunk by the plenum-pneumatic process for the piers of a bridge over the Schuylkill River at South Street, Philadelphia. There were three piers, one of which was a pivot pier. There were two cylinders, 8 feet in diameter and 82 feet long, sunk through 22 feet of water and 30 feet of "sand and tough compact mud intermingled with bowlders;" two cylinders, 8 feet in diameter and 57 feet long, sunk through 22 feet of water and 5 feet of soil as above; one cylinder, 6 feet in diameter and 64 feet long, sunk through 22 feet of water and 18 feet of soil as above; and 8 columns, 4 feet in diameter and aggregating 507 feet, sunk through 22 feet of water and 18 feet of soil as above. A 10-foot section of the 8-foot cylinder weighed 14,600 pounds, of the 6-foot, 10,800 pounds, and of the 4-foot, 6,800 pounds. The cylinders rested upon bed-rock, and were bolted to

* Data by courtesy of Sooysmith & Co., contracting engineers for the pneumatic foundations.

it. The actual cost to the contractor, exclusive of tools and machinery, was as in Table 34 (page 304).

TABLE 33.

COST OF PNEUMATIC FOUNDATIONS OF BLAIR BRIDGE.*

ITEMS.	NUMBER OF THE PIER.			
	I.	II.	III.	IV.
Total distance caisson was lowered after completion	55.6 ft.	54.5 ft.	56.2 ft.	68.5 ft.
Final depth of cutting edge below surface of water	51.9 "	52.3 "	53.4 "	57.0 "
" " " " " " mud line	47.7 "	51.0 "	49.4 "	54.7 "
Caisson and filling, cost of	$11,753.51	$12,886.56	$13,819.34	$11,252.45
" " " " " per cubic yard	14.31	15.12	16.74	13.77
Crib and filling, cost of	7,368.16	....	....	6,303.46
" " " " " per cubic yard	8.85	....	....	7.59
Air-lock, shafts, etc., cost of	1,481.60	1,567.42	1,536.80	1,521.08
Sinking caisson, cost of, including erection and removal of machinery	5,772.52	5,629.87	6,888.16	7,084.26
Sinking caisson, cost of, per cubic foot of displacement below position of cutting edge when caisson was completed	8.0 cts.	8.0 cts.	9.5 cts.	7.1 cts.
Sinking caisson, cost of, per cubic foot of displacement below surface of water	8.6 "	8.3 "	9.9 "	9.6 "
Sinking caisson, cost of, per cubic foot of displacement below mud line	9.3 "	8.5 "	10.8 "	10.0 "
Total cost † of foundation	$26,375.79	$19,583.35	$22,244.30	$26,161.25
" " " " per cubic yard	15.98	23.87	27.08	15.85

Average cost † of the foundations, per cubic yard. ....$20.70.

465. "Excavation in the Brooklyn caisson ‡ cost for labor only, including the men on top, about $5.25 per cubic yard [19 cents per cubic foot]. Running the six air-compressors added to this $3.60 per hour, or about 47 cents per yard; lights added $0.56 more; and these with other contingencies nearly equaled the cost of labor. The great cost was due to the excessive hardness of the material over much of the surface, the caisson finally resting, for nearly its whole extent, on a mass of bowlders or hardpan. The concrete in the caisson cost, for every expense, about

* Compiled from the report of Geo. S. Morison, chief engineer of the bridge.

† Exclusive of engineering expenses and cost of tools, machinery, and buildings. In a note to the author, Mr. Morison, the engineer of the bridge says: "It is impossible to divide the buildings, tools, and engineering expenses between the substructure and other portions of the work. The bulk of the items of tools and machinery [$12,369.88], however, relates to the foundations." The engineering expenses and buildings were nearly 3 per cent. of the total cost of the entire bridge. The cost of tools and machinery was equal to a little over 13 per cent. of the cost of the foundations as above. Including these items would add nearly one sixth to the amounts in the last three lines.

‡ For a brief description, see § 458.

TABLE 34.

COST OF PNEUMATIC PILES AT PHILADELPHIA IN 1869–72.*

ITEMS OF EXPENSE.	DIAMETER OF CYLINDERS.		
	4-ft.	6-ft.	8-ft.
Cost of cast iron, @ $59.50 per ton	$11,239.36	$2,053.75	$13,577.90
" " bolts, @ 9¾ cents per lb	489.84	93.31	670.02
" " grouted rubble masonry (exclusive of labor), @ $5.40 per cu. yd	1,266.79	358.40	2,779.97
" " sinking, and laying masonry	6,693.50	911.88	9,036.51
Total cost of the cylinders in place	$19,689.49	$3,417.34	$26,064.40
Cost of iron per lineal foot of cylinder	$23.10	$33.54	$51.25
" " materials for masonry per lineal foot of cylinder	2.50	5.60	10.00
" " sinking and laying masonry per lineal foot of cylinder	13.20	14.25	32.51
Total cost,† per lineal foot, of cylinder in place	$38.80	$53.39	$93.76

$15.50 per cubic yard. The caisson and filling together aggregated 16,898 cubic yards; and the approximate cost per yard for every expense was $20.71."‡ The foundation therefore cost about $30 per cubic yard.

The pneumatic foundations for the channel piers of the bridge over the Missouri at Plattsmouth, Neb., cost as follows: One foundation, consisting of a caisson 50 ft. long, 20 ft. wide, and 15.5 ft. high, surmounted by a crib 14.15 ft. high, sunk through 13 ft. of water and 20 ft. of soil, cost $19.29 per cubic yard of net volume. Another, consisting of a caisson 50 ft. long, 20 ft. wide, and 15.5 ft. high, surmounted by a crib 36.25 ft. high, sunk through 10 ft. of water and 44 ft. of soil, cost $14.45 per cubic yard of net volume. §

466. *European Examples.* The following¶ is interesting as showing the cost of pneumatic work in Europe:

"At Moulins, cast-iron cylinders, 8 feet 2½ inches in diameter, with a filling of concrete and sunk 33 feet below water into marl, cost $62.94 per lineal foot, or $29.71 for the iron work, and $33.23

* Compiled from an article by D. McN. Stauffer, engineer in charge, in Trans. Am. Soc. of C. E., vol. vii. pp. 287–309.

† Exclusive of tools and machinery.

‡ F. Collingwood, assistant engineer Brooklyn bridge, in Trans. Am. Soc. of C. E.

§ Compiled from the report of Geo. S. Morison, chief engineer of the bridge.

¶ By Jules Gaudard, as translated from the French by L. F. Vernon-Harcourt for the Proceedings of the Institute of Civil Engineers (London).

for sinking and concrete. At Argenteuil, with cylinders 11 feet 10 inches in diameter, the sinking alone cost $42.12 per lineal foot [nearly $10 per cubic yard], where a cylinder was sunk 53¼ feet in three hundred and ninety hours. [The total cost of this foundation was $34.09 per cubic yard, see table on page 310.] At Orival, where a cylinder was sunk 49 feet in twenty days, the cost of sinking was $36.83 per lineal foot. At Bordeaux, with the same-sized cylinders, a gang of eight men conducted the sinking of one cylinder, and usually 34 cubic yards were excavated every twenty-four hours. The greatest depth reached was 55¾ feet below the ground, and 71 feet below high water. In the regular course of working, a cylinder was sunk in from nine to fifteen days, and the whole operation, including preparations and filling with concrete, occupied on the average 25 days. One cylinder, or a half pier, cost on the average $11,298.40, of which $1,461 was for sinking. M. Morandière estimates the total cost of a cylinder sunk like those at Argenteuil, to a depth of 50 feet, at $7,012.80.

467. "Considering next the cost of piers of masonry on wrought-iron caissons of excavation, the foundations of the Lorient viaduct over the Scorff cost the large sum of $24.11 per cubic yard, owing to difficulties caused by the tides, the labor of removing the bowlders from underneath the caisson, and the large cost of plant for only two piers. The foundation of the Kehl bridge cost still more, about $28.23 * per cubic yard; but this can not be regarded as a fair instance, being the first attempt [see § 429] of the kind.

"The foundations of the Nantes bridges, sunk 56 feet below low-water level, cost about $14.84 per cubic yard. The average cost per pier was as follows:

Caisson (41 feet 4 inches by 14 feet 5 inches), 50 tons of wrought iron @ $116.88	$5,844
Coffer-dam, 3 tons of wrought iron @ $58.44	175
Excavation, 916 cubic yards @ $4.47	4,091
Concrete	4,188
Masonry, plant, etc.	1,870
Average cost per pier	$16,168

"One pier of the bridge over the Meuse at Rotterdam, with a

* Notice the slight inconsistency between this quantity and the one in the third line from the last of the table on page 310, both being from the same article.

caisson of 222 tons and a coffer-dam casing of 94 tons, and sunk 75 feet below high water, cost $70,858, or $13.97 per cubic yard.

"The Vichy bridge has five piers built on caissons 34 feet by 13 feet, and two abutments on caissons 26 feet by 24 feet. The foundations were sunk 23 feet in the ground, the upper portion consisting of shingle and conglomerated gravel, and the last 10 feet of marl. The cost of the bridge was as follows:

Interest for eight months, and depreciation of plant worth $19,480..	$3,896
Cost of preparations, approach bridge, and staging..................	4,904
Caissons (seven), 150¼ tons @ $113.88..............................	17,108
Sinking..	9,823
Concrete and masonry..	5,303
Contractor's bonus and general expenses...........................	6,107
Total cost of five foundations.........................	$47,141

The cost per cubic yard of the foundation below low water was $16.69, of which the sinking alone cost $3.50 in gravel, and $4.37 in marl.

"At St. Maurice, the cost per cubic yard of foundation was $15.94, exclusive of staging."

468. Conclusion. Except in very shallow or very deep water, the compressed-air process has almost entirely superseded all others. The following are some of the advantages of this method. 1. It is reliable, since there is no danger of the caisson's being stopped, before reaching the desired depth, by sunken logs, bowlders, etc., or by excessive friction, as in dredging through tubes or shafts in cribs. 2. It can be used regardless of the kind of soil overlying the rock or ultimate foundation. 3. It is comparatively rapid, since the sinking of the caisson and the building up of the pier go on at the same time. 4. It is comparatively economical, since the weight added in sinking is a part of the foundation and is permanent, and the removal of the material by blowing out or by pumping is as uniform and rapid at one depth as at another,—the cost only being increased somewhat by the greater depth. 5. This method allows ample opportunity to examine the ultimate foundation, to level the bottom, and to remove any disintegrated rock. 6. Since the rock can be laid bare and be thoroughly washed, the concrete can be commenced upon a perfectly clean surface; and hence there need be no question as to the stability of the foundation.

ART. 5. THE FREEZING PROCESS.

469. PRINCIPLE. The presence of water has always been the great obstacle in foundation work and in shaft sinking, and it is only very recently that any one thought of transforming the liquid soil into a solid wall of ice about the space to be excavated. The method of doing this consists in inclosing the site to be excavated, by driving into the ground a number of tubes through which a freezing mixture is made to circulate. These consist of a large tube, closed at the lower end, inclosing a smaller one, open at the lower end. The freezing mixture is forced down the inner tube, and rises through the outer one. At the top, these tubes connect with a reservoir, a refrigerating machine, and a pump. The freezing liquid is cooled by an ice-making machine, and then forced through the tubes until a wall of earth is frozen around them of sufficient thickness to stand the external pressure, when the excavation can proceed as in dry ground.

470. HISTORY. This method was invented by F. H. Poetsch, M.D., of Aschersleben, Prussia, in 1884. It has been applied in but three cases. The first was at the Archibald colliery, near Schweidlingen, Prussia, where a vein of quicksand, 20 feet thick, was encountered at a depth of about 150 feet below the surface. Here twenty-three pipes were used, and 35 days consumed in the freezing process, under local difficulties. The second was at the Centrum mine, near Berlin, where about 107 feet of quicksand, etc., was penetrated. Engineers had been baffled for years in their attempts to sink a shaft here; but in 33 days Mr. Poetsch had, with only 16 freezing tubes, secured a 6-foot wall of ice around the shaft area, and the shaft was excavated and curbed without difficulty. The third piece of work was at the Eimilia mine, Fensterwalde, Austria, in 1885, where an 8½-foot shaft was sunk through 115 feet of quicksand.*

471. DETAILS OF THE PROCESS. In the last case mentioned above, "12 circulatory tubes were used, sunk in a circle about 14 feet in diameter, from 12 to 15 days being required to sink them a depth of about 100 feet. The outside tubes were 8½ inches in

* As this volume is going through the press, this method is being applied in two places in this country—Iron Mountain, Mich., and Wyoming, Penn.—in sinking mine-shafts.

diameter, and made of plate iron 0.15 inch thick. The tubes were sunk by aid of the water-jet. They were given a very slight inclination outward at the bottom to avoid any deviation in sinking that might interfere with the line of the shaft. The freezing liquid employed was a solution of chloride of calcium, which congeals at a temperature of −35° C. (−31° F.). The circulation of the liquid through the tubes was secured by a small pump with a piston 6 inches in diameter and a 12-inch stroke. At the beginning of the operation, this pump made 30 double strokes per minute, which was equivalent to the passage of 0.6 gallon of the liquid through each tube per minute; at the end of the operation, when it was only necessary to maintain the low temperature, the pump strokes were reduced to 15 per minute. The refrigerating machine employed was one of a model guaranteed by the maker to produce 1,100 pounds of ice per hour. The motive-power was supplied by a small engine of about 5 horse-power. The ammoniac pump had a piston 2.8 inches in diameter and a 9.2-inch stroke, and made 30 strokes per minute. The pressure maintained was about 10 atmospheres. The quantity of ammoniacal liquid necessary to charge the apparatus was 281 gallons; and under normal conditions the daily consumption of this liquid was 0.78 gallon.

"The actual shaft excavation was commenced 53 days after the freezing apparatus had been set in motion. The freezing machine was in operation 240 days. The work was done without difficulty, and a progress of 1.64 feet per day was made. The timbering was very light, but no internal pressure of any kind was observed. The brick masonry used for finally lining the shaft was about 11 inches thick. When the shaft was finished, the tubes were withdrawn without difficulty, by circulating through them a hot, instead of a cold, solution of the chloride of calcium, thus thawing them loose from the surrounding ice. The tubes were entirely uninjured, and could be used again in another similar operation.

472. "The material in the above plant is estimated to have cost $15,000, and $4,800 more for mounting and installation. The daily expense of conducting the freezing process is estimated at $11. The total expense for putting down the shaft is estimated at $128.66 per linear foot."* The last is equivalent to about $2.25 per cubic foot.

* *Engineering News*, vol. xiv. pp. 24, 25, translated from *Le Génie Civil* of June 13, 1885.

473. Modification for Foundations under Water. For sinking foundations under water, two methods of applying this process have been proposed. One of these consists in combining the pneumatic and freezing processes. A pneumatic caisson is to be sunk a short distance into the river-bed, and then the congealing tubes are applied, and the entire mass between the caisson and the rock is frozen solid. When the freezing is completed, the caisson will be practically sealed against the entrance of water, and the air-lock can be removed and the masonry built up as in the open air.

The other method consists in sinking an open caisson to the river-bed, and putting the freezing tubes down through the water. When the congelation is completed, the water can be pumped out and the work conducted in the open air.

474. Advantages Claimed. It is claimed for this process that it is expeditious and economical, and also that it is particularly valuable in that it makes possible an accurate estimate of the total cost before the work is commenced,—a condition of affairs unattainable by any other known method in equally difficult ground. It has an advantage over the pneumatic process in that it is not limited by depth. It can be applied horizontally as well as vertically, and hence is specially useful in sub-aqueous tunneling, particularly in soils which, with compressed air, are treacherous.

475. Difficulties Anticipated. So far it has been used only in sinking shafts for mines. Two difficulties are anticipated in applying it to sink foundations for bridge piers in river beds; viz., (1) the difficulty in sinking the pipes, owing to striking sunken logs, bowlders, etc.; and (2) the possibility of encountering running water, which will thaw the ice-wall. These difficulties are not insurmountable, but experience only can demonstrate how serious they are.

476. Cost. See § 472, and compare with table on page 310.

Art. 6. Comparison of Methods.

477. The following comparison of the different methods is from an article by Jules Gaudard on Foundations, as translated by L. F. Vernon-Harcourt for the proceedings of the Institute of Civil Engineers (London). Except as showing approximate relative costs in Europe, it is not of much value, owing to improvement made since the article was written, to the differences between European and

American practice, and to differences in cost of materials in the two countries.

478. "M. Croizette Desnoyers has framed a classification of the methods of foundations most suitable for different depths, and also an estimate of the cost of each. These estimates, however, must be considered merely approximate, as unforeseen circumstances produce considerable variations in works of this nature.

TABLE 35.

COST OF VARIOUS KINDS OF FOUNDATIONS IN EUROPE.

KIND OF FOUNDATION.	DEPTH IN FEET.		COST PER CUBIC YARD.	
	Min.	Max.	Min.	Max.
On piles after compression of the ground, shallow depth	20	33	$2.92	$4.39
" " " " " " " greater depth	33	50	4.39	7.30
By sinking wells	33	50	7.30	9.00
By pumping	......	20	2.92	4.39
" " under favorable circumstances	26	33	4.39	13.39
" " " unfavorable circumstances	26	33	14.85	17.77
On concrete under water, small amount of silt	20	33	4.37	9.00
" " " " large " " "	20	33	9.00	11.93
By means of compressed air* under favorable circumstances	......	......	13.39	16.17
" " " " " " unfavorable circumstances:				
Lorient viaduct	[50 ft.]		$24.11	
Kehl bridge †	[70 "]		29.71	
Argenteuil bridge	[50 "]		34.09	
Bordeaux bridge	....		40.17	

* See also §§ 466–67.

† See foot-note on page 305.

"When the foundations consist of disconnected pillars or piles, the above prices must be applied to the whole cubic content, including the intervals between the parts; but of course at an equal cost solid piers are the best.

479. "For pile-work foundations the square yard of base is probably a better unit than the cubic yard. Thus the foundations of the Vernon bridge, with piles from 24 to 31 feet long, and with cross-timbering, concrete, and caisson, cost $70 per square yard of base. According to estimates made by M. Picquenot, if the foundations had been put in by means of compressed air, the cost would have been $159.64; with a caisson, not water-tight, sunk down, $66.27; with concrete poured into a space inclosed with sheeting, $62.23; and by pumping, $83.56 per square yard of base."

PART IV.

MASONRY STRUCTURES.

CHAPTER XIII.

MASONRY DAMS.

480. It is not the intention here to discuss every feature of masonry dams; that has been done in the special reports and articles referred to at the end of this chapter. The fundamental principles will be considered, particularly with reference to their application in the subsequent study of retaining walls, bridge abutments, bridge piers, and arches. The discussions of this chapter are applicable to masonry dams, reservoir walls, or to any wall which counteracts the pressure of water mainly by its weight.

There are two ways in which a masonry dam may resist the thrust of the water; viz., (1) by the inertia of its masonry, and (2) as an arch. 1. The horizontal thrust of the water may be held in equilibrium by the resistance of the masonry to sliding forward or to overturning. A dam which acts in this way is called a *gravity dam*. 2. The thrust of the water may be resisted by being transmitted laterally to the side-hills (abutments) by the arch-like action of the masonry. A dam which acts in this way is called an *arched dam*.

Only two dams of the pure arch type have ever been built. The almost exclusive use of the gravity type is due to the uncertainty of our knowledge concerning the laws governing the stability of masonry arches. This chapter will be devoted mainly to gravity dams, those of the arch type being considered only incidentally. Arches will be discussed fully in Chapter XVIII.

ART. 1. STABILITY OF GRAVITY DAMS.

481. PRINCIPLES. By the principles of hydrostatics we know (1) that the pressure of a liquid upon any surface is equal to the weight of a volume of the liquid whose base is the area of the immersed surface and whose height is the vertical distance of the center of gravity of that surface below the upper surface of the water; (2) that this pressure is always perpendicular to the pressed surface; and (3) that, for rectangular surfaces, this pressure may be considered as a single force applied at a distance below the upper surface of the liquid equal to $\frac{2}{3}$ of the depth.

482. A gravity dam may fail (1) by sliding along a horizontal joint, or (2) by overturning about the front of a horizontal joint, or (3) by crushing the masonry, particularly at the front of any horizontal joint. However, it is admitted that by far the greater number of failures of dams is due to defects in the foundation. The method of securing a firm foundation has already been discussed in Part III; and, hence, this subject will be considered here only incidentally. There is not much probability that a dam will fail by sliding forward, but it may fail by overturning or by the crushing of the masonry. These three methods of failure will be considered separately and in the above order.

483. In the discussions of this article it will be necessary to consider only a section of the wall included between two vertical planes—a unit distance apart—perpendicular to the face of the wall, and then so arrange this section that it will resist the loads and pressure put upon it; that is, it is sufficient, and more convenient, to consider the dam as only a unit, say 1 foot, long.

484. NOMENCLATURE. The following nomenclature will be used throughout this chapter:

H = the horizontal pressure, in pounds, of the water against a section of the back of the wall 1 foot long and of a height equal to the height of the wall.

W = the weight, in pounds, of a section of the wall 1 foot long.

w = the weight, in pounds, of a cubic foot of the masonry.

h = the height, in feet, of the wall; *i. e.*, $h = E F$, Fig. 68.

l = the length of the base of the cross section; *i. e.*, $l = A B$, Fig. 68.

t = the width of the wall on top; *i. e.*, $t = D E$, Fig. 68.

b = the batter of the wall, *i. e.*, the inclination of the surface per foot of rise—b' being used for the batter of the up-stream face and b_1 for that of the down-stream face.

$\bar{x} = A\,C$ = the distance from the down-stream face of any joint to the point in which a vertical through the center of gravity of the wall pierces the plane of the base.

d = the distance the center of pressure deviates from the center of the base.

62.5 = the weight, in pounds, of a cubic foot of water.

485. Stability against Sliding. The horizontal pressure of the water tends to slide the dam forward, and is resisted by the friction due to the weight of the wall.

486. Sliding Force. The horizontal pressure of the water against an elementary section of the wall, by principle (1) of § 481, is equal to the area of the section multiplied by half the height of the wall, and that product by the weight of a cubic unit of water; or

$$H = h \times 1 \times \tfrac{1}{2}\,h \times 62.5 = 31.25\,h^2. \quad . \quad . \quad . \quad (1)$$

Notice that H is the same whether the pressed area is inclined or vertical; that is to say, H is the horizontal component of the total pressure on the surface.

487. Resisting Forces. The weight of an elementary section of the wall is equal to the area of the vertical cross section multiplied by the weight of a cubic unit of the masonry. The area of the cross section, $A\,B\,E\,D$, Fig. 68, equals

$$E\,F \times D\,E + \tfrac{1}{2}\,E\,F \times F\,B + \tfrac{1}{2}\,D\,G \times A\,G$$
$$= h\,t + \tfrac{1}{2}\,h^2\,b' + \tfrac{1}{2}\,h^2\,b_1 \quad . \quad . \quad . \quad (2)$$

Then the weight of the elementary section of the wall is

$$W = w\,(h\,t + \tfrac{1}{2}\,h^2\,b' + \tfrac{1}{2}\,h^2\,b_1) \quad . \quad (3)$$

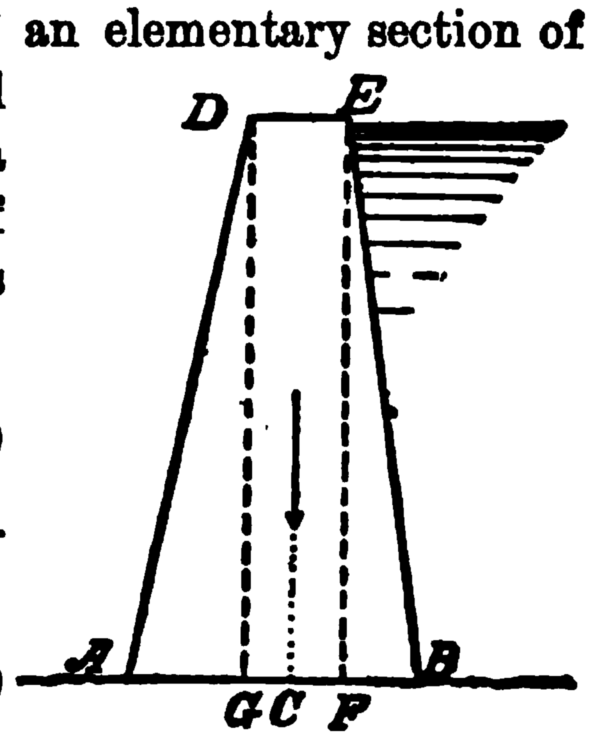

FIG. 68.

The vertical pressure of the water on the inclined face increases the pressure on the foundation, and, consequently, adds to the resistance against sliding. The vertical pressure on $E\,B$ is equal to the horizontal projection of that area multiplied by the distance of the center of gravity of the surface below the top of the water and by the weight of a cubic unit of

water, or, the vertical pressure $= FB \times 1 \times \frac{1}{2} h \times 62.5 = h\, b' \times \frac{1}{2} h \times 62.5 = 31.25\, h^2\, b'$.

488. If the earth rests against the heel of the dam (the bottom of the inside face), it will increase the pressure on the foundation, since earth weighs more than water ; on the other hand, the horizontal pressure of the earth will be a little greater than that of an equal height of water. However, since the net resistance with the earth upon the heel of the wall is greater than with an equal depth of water, it will be assumed that the water extends to the bottom of the wall.

If the water finds its way under and around the foundation of the wall, even in very thin sheets, it will decrease the pressure of the wall on the foundation, and, consequently, decrease the stability of the wall. The effective weight of the submerged portion of the wall will be decreased $62\frac{1}{2}$ lbs. per cu. ft. However, the assumption that water in hydrostatic condition finds its way under or into a *dam* is hardly admissible; hence the effect of buoyancy will not be considered.*

489. Co-efficient of Friction. The values of the co-efficient of friction most frequently required in masonry computations are given in the table on page 315. There will be frequent reference to this table in subsequent chapters ; and therefore it is made more full than is required in this connection. The values have been collected from the best authorities, and are believed to be fair averages. See also the table on page 276.

490. Condition for Equilibrium. In order that the wall may not slide, it is necessary that the product found by multiplying the co-efficient of friction by the sum of the weight of the wall and the vertical pressure of the water shall be greater than the horizontal pressure of the water. That is to say, in order that the dam may not slide it is necessary that $\mu\,(W + 31.25\, h^2\, b')$ shall be greater than H; or, in mathematical language,

$$\mu > \frac{H}{W + 31.25\, h^2\, b'} > \frac{31.25\, h^2}{w\,(h\,t + \frac{1}{2}\, h^2\, b' + \frac{1}{2}\, h^2\, b_1) + 31.25\, h^2\, b'},$$

* Since the above was written, Jas. B. Francis presented a paper (May 16, 1888) before the American Society of Civil Engineers, which seems to show that water pressure is communicated, almost undiminished, through a layer of Portland cement mortar (1 part cement and 2 parts sand) 1 foot thick.

TABLE 36.

CO-EFFICIENTS OF FRICTION FOR DRY MASONRY.

DESCRIPTION OF THE MASONRY.	CO-EFFICIENT.
Soft limestone on soft limestone, both well dressed	0.75
Brick-work on brick-work, with slightly damp mortar	0.75
Hard brick-work on hard brick-work, with slightly damp mortar.	0.70
Point-dressed granite on like granite	0.70
" " " " well-dressed granite	0.65
Common brick on common brick	0.65
" " " hard limestone	0.65
Hard limestone on hard limestone, with moist mortar	0.65
Beton blocks (pressed) on like beton blocks	0.65
Fine-cut granite on pressed " "	0.60
Well-dressed granite on well dressed granite	0.60
Polished limestone on polished limestone	0.60
Well-dressed granite on like granite, with fresh mortar	0.50
Common brick on common brick, with wet mortar	0.50
Polished marble on common brick	0.45
Point-dressed granite on gravel	0.60
" " " " dry clay	0.50
" " " " sand	0.40
" " " " moist clay	0.33
Wrought iron on well-dressed limestone	0.50
" " " hard, well-dressed limestone, wet	0.25
Oak, flatwise, on limestone	0.65
" endwise, on limestone	0.40

which reduced becomes

$$\mu > \frac{62.5\, h}{w\,(2\, t + h\,(b' + b_1) + 62.5\, h\, b'} . \quad . \quad . \quad (4)$$

The weight of a cubic foot of masonry, w, varies between 125 lbs. for concrete or poor brick-work, and 160 lbs. for granite ashlar. Dams are usually built of rubble, which weighs about 150 lbs. per cu. ft. To simplify the formula, we will assume that the masonry weighs 125 lbs. per cu. ft.; *i. e.*, that the weight of a cubic foot of masonry is twice that of water. This assumption is on the safe side, whatever the kind of masonry.* Making this substitution in (4),

* Increased safety generally requires increased cost of construction, and hence it is not permissible to use approximate data simply because the error is on the side toward safety. It will be shown that there is no probability of any dam's failing by sliding, and that the size, and consequently the volume and cost, are determined by the dimensions required to prevent crushing and overturning; hence this approximation involves no increase in the cost.

and dropping the term $2\,h\,b'$ in the denominator,—since to do so makes but little difference and is on the safe side,—we have

$$\mu > 0.5\,\frac{h}{2\,t + h\,(b' + b_1)} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Other things being the same, the thinner the wall at the top, the easier it will slide. If the section of the wall is a triangle, *i. e.*, if $t = 0$, then by equation (5) we see that the dam is safe against sliding when

$$\mu > 0.5\,\frac{1}{(b' + b_1)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

An examination of the table on page 315 shows that there is no probability that the co-efficient of friction will be less than 0.5; and inserting this value of μ in (6) shows that sliding can not take place if $(b' + b_1) >$ or $= 1$. To prevent overturning, the sum of the batters are usually $=$ or > 1 (see Fig. 72, p. 328); and, besides, a considerable thickness at the top (see § 509) is needed to resist the shock of waves, etc. Hence there is no probability of the dam's failing by sliding forward. Further, the co-efficient of friction in the table on page 315 takes no account of the cohesion of the mortar, which may have a possible maximum value, for best Portland mortar, of 36 tons per sq. ft. (500 lbs. per sq. in.); and this gives still greater security. Again, the earth on, and also in front of, the toe of the wall adds greatly to the resistance against sliding. Finally, it is customary to build masonry dams of uncoursed rubble (§§ 213–17), to prevent the bed-joints from becoming channels for the leakage of water; and hence the stones are thoroughly interlocked,—which adds still further resistance. Therefore it is certain that there is no danger of any masonry dam's failing by sliding forward under the pressure of still water.

491. It has occasionally happened that dams and retaining walls have been moved bodily forward, sliding on their base; but such an occurrence is certainly unusual, and is probably the result of the wall's having been founded on an unstable material, perhaps on an inclined bed of moist and uncertain soil. In most that was said in Part III concerning foundations, it was assumed that the founda-

tion was required to support only a vertical load. When the structure is subjected also to a lateral pressure, as in dams, additional means of security are demanded to prevent lateral yielding.

When the foundation rests upon piles a simple expedient is to drive piles in front of and against the edge of the bed of the foundation; but obviously this is not of much value except when the piles reach a firmer soil than that on which the foundation directly rests. If the piles reach a firm subsoil, it will help matters a little if the upper and more yielding soil is removed from around the top of the pile, and the place filled with broken stone, etc. Or a wall of piles may be driven around the foundation at some distance from it, and timber braces or horizontal buttresses of masonry may be placed at intervals from the foundation to the piles. A low masonry wall is sometimes used, instead of the wall of piles, and connected with the foot of the main wall by horizontal buttresses, whose feet, on the counter-wall, are connected by arches in a horizontal plane in order to distribute the pressure more evenly.

In founding a dam upon bed-rock, the resistance to sliding on the foundation may be greatly increased by leaving the bed rough; and, in case the rock quarries out with smooth surfaces, one or more longitudinal trenches may be excavated in the bed of the foundation, and afterwards be filled with the masonry.

In the proposed Quaker Bridge dam the maximum horizontal thrust of the water is equal to 0.597 of the weight of the masonry.

492. STABILITY AGAINST OVERTURNING. The horizontal pressure of the water tends to tip the wall forward about the front of any joint, and is resisted by the moment of the weight of the wall. For the present, it will be assumed that the wall rests upon a rigid base, and therefore can fail only by overturning as a whole.

The conditions necessary for stability against overturning can be completely determined either by considering the moments of the several forces, or by the principle of resolution of forces. In the following discussion the conditions will be first determined by moments, and afterward by resolution of forces.

493. A. BY MOMENTS. The Overturning Moment. The pressure of the water is perpendicular to the pressed surface. If the water presses against an inclined face, then the pressure makes the same angle with the horizontal that the surface does with the vertical. Since there is a little difficulty in finding the arm of this force, it is

more convenient to deal with the horizontal and vertical components of the pressure.

The horizontal pressure of the water can be found by equation (1), page 313. The arm of this force is equal to $\frac{1}{3}\ h$ (principle 3, § 481). Hence the moment tending to overturn the wall is equal to

$$\tfrac{1}{3}\ H\ h = \tfrac{1}{3}\ 31.25\ h^3 = 10.42\ h^3, \quad . \quad . \quad . \quad . \quad . \qquad (7)$$

which, for convenience, represent by M_1.

494. The Resisting Moments. The forces resisting the overturning are (1) the weight of the wall and (2) the vertical pressure of the water on the inclined face.

The weight of the wall can be computed by equation (3), page 313. It acts vertically through the center of gravity of the cross section.

The center of gravity can be found algebraically or graphically. There are several ways in each case, but the following graphical solution is the simplest. In Fig. 69, draw the diagonals $D\ B$ and $A\ E$, and lay off $A\ J = E\ I$; then draw $D\ J$, and mark the middle of it Q. The center of gravity, O, of the area $A\ B\ E\ D$ is at a distance from Q towards B equal to $\frac{1}{3}\ Q\ B$. This method is applicable to any four-sided figure.

The position of the center of gravity can also be found algebraically by the principle that the moment of the entire mass about any point, as A, is equal to the moment of the part $A\ D\ G$, *plus* the moment of the portion $D\ E\ F\ G$, *plus* the moment of the part $E\ B\ F$,—all about the same point, A. Stating this principle algebraically gives

G F

FIG. 69.

$$\tfrac{1}{2}\ h\ b_1\ (\tfrac{2}{3}\ h^2\ b_1) + h\ t\ (\tfrac{1}{2}\ t + h\ b_1) + \tfrac{1}{2}\ h^2\ b'\ (\tfrac{1}{3}\ h\ b' + t + h\ b')$$
$$= (\tfrac{1}{2}\ h^2\ b' + h\ t + \tfrac{1}{2}\ h^2\ b_1)\ \bar{x}, \quad . \quad . \quad . \quad . \quad . \quad . \qquad (8)$$

in which $\bar{x} =$ the distance $A\ C$. Solving (8) gives

$$\bar{x} = \frac{\tfrac{1}{3}\ h^2\ (b_1^2 + 2\ b'^2) + h\ t\ b_1 + \tfrac{1}{2}\ t\ (t + b'h)}{\tfrac{1}{2}\ h\ (b' + b_1) + t} \quad . \quad . \quad . \qquad (9)$$

The arm of the weight is $A\ C\ (= \bar{x})$, and therefore the moment is

$$W \times A\ C = w\,[h\,t + \tfrac{1}{2} h^2 (b' + b_1)]\,\bar{x}, \quad . \quad . \quad . \quad (10)$$

which, for convenience, represent by M_2.

495. The vertical pressure of the water on the inclined face, $E\ B$, has been computed in § 487, which see. This force acts vertically between F and B, at a distance from B equal to $\frac{1}{3}\ F\ B$; the arm of this force is $A\ B - \frac{1}{3}\ F\ B = l - \frac{1}{3}\ h\ b' = h\ b_1 + t + \frac{2}{3}\ h\ b'$. Therefore, the moment of the vertical pressure on the inclined face is

$$31.25\ h^2\ b'\ (h\ b_1 + t + \tfrac{2}{3}\ h\ b'), \quad . \quad . \quad . \quad . \quad (11)$$

which, for convenience, represent by M_3. Of course, if the pressed face is vertical, M_3 will be equal to zero.

496. The moment to resist overturning is equal to the sum of (10) and (11) above, or $M_2 + M_3$.

The moment represented by the sum of M_1 and M_3 can be determined directly by considering the pressure of the water as acting perpendicular to $E\ B$ at $\frac{1}{3}\ E\ B$ from B; the arm of this force is a line from A perpendicular to the line of action of the pressure. If the cross section were known, it would be an easy matter to measure this arm on a diagram; but, in designing a dam, it is necessary to know the conditions requisite for stability before the cross section can be determined, and hence the above method of solution is the better.

497. Condition for Equilibrium. In order that the wall may not turn about the front edge of a joint, it is necessary that the overturning moment, M_1, as found by equation (7), shall be ~~greater~~ than the sum of the resisting moments, M_2 and M_3, as found by equations (10) and (11); or, in other words, the factor against overturning $= \dfrac{M_2 + M_3}{M_1}$. . . . . (12)

498. Factor of Safety against Overturning. In computing the stability against overturning, the vertical pressure of the water against the inside face is frequently neglected; *i. e.*, it is assumed that M_3, as above, is zero. This assumption is always on the safe side. Computed in this way, the factor of safety against overturning for the proposed Quaker Bridge dam, which when completed

will be considerably the largest dam in the world, varies between 2.07 and 3.68. Krantz,* who included the vertical component in his computations, considers a factor of 2.5 to 5.55 as safe, the larger value being for the largest dam, owing to the more serious consequences of failure. The greater the factor of safety provided for, the greater is the first cost; and the less the factor of safety, the greater the expense of maintenance, including a possible reconstruction of the structure.

499. B. By Resolution of Forces. In Fig. 70, K is the center of pressure of the water on the back of the wall. $KB = \frac{1}{3} EB$. o is the center of gravity of the wall,—found as already described. Through K draw a line, Ka, perpendicular to EB; through o draw a vertical line oa. To any convenient scale lay off ab equal to the total pressure of the water against $E\ B$, and to the same scale make af equal to the weight of an elementary section of the wall. Complete the parallelogram $abef$. The diagonal ae intersects the base of the wall at N.

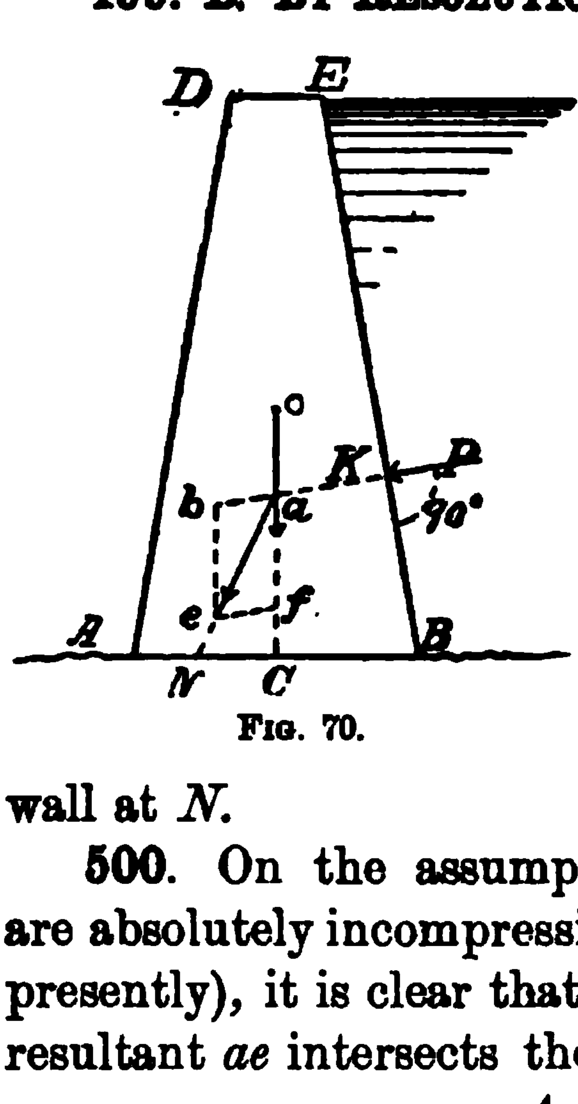

Fig. 70.

500. On the assumption that the masonry and foundation are absolutely incompressible (the compressibility will be considered presently), it is clear that the wall will not overturn as long as the resultant ae intersects the base AB between A and B. The factor against overturning is $\frac{A\ C}{N\ C}$, which is the equivalent of equation (12).

The wall can not slide horizontally on the base, when the angle NaC is less than the angle of repose, *i. e.*, when $\tan NaC$ is less than the co-efficient of friction. The factor against sliding is equal to the co-efficient of friction divided by $\tan NaC$, which is only another way of stating the conclusion drawn from equation (4), page 315.

501. Stability against Crushing. The preceding discussion of the stability against overturning is on the assumption that the masonry does not crush. This method of failure will now be con-

* "Study of Reservoir Walls," Mahan's translation, p. 53.

sidered. When the reservoir is empty, the pressure tending to produce crushing is the weight of the dam alone, which pressure is distributed uniformly over the horizontal area of the wall. When the reservoir is full, the thrust of the water modifies the distribution of the pressure, increasing the pressure at the front of the wall and decreasing it at the back. We will now determine the law of the variation of the pressure.

Let A B, Fig. 71, represent the base of a vertical section of the dam; or A B may represent the rectangular base (whose width is a unit) of any two bodies which are pressed against each other by any forces whatever.

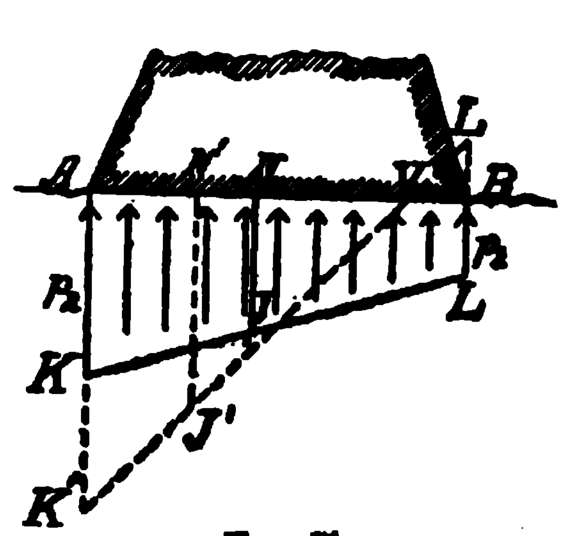

FIG. 71.

M = the resulting moment (about A) of all the external forces. In the case of a dam, $M = M_1 - M_2$,—see equations (7) and (11).

W = the total normal pressure on A B.
In the case of a dam, W = the weight of the masonry.

P = the maximum pressure, per unit of area, at A.

p = the change in unit pressure, per unit of distance, from A towards B.

x = any distance from A towards B.

$P - p\,x$ = the pressure per unit at a distance x from A.

Y = a general expression for a vertical force.

The remainder of the nomenclature is as in § 484, page 312.

Taking moments about A gives

$$M - W\bar{x} + \int_0^l (P - p\,x)\,dx \,.\, x = 0; \quad . \quad . \quad . \quad (13)$$

$$M - W\bar{x} + \tfrac{1}{2} P l^2 - \tfrac{1}{3} p\, l^3 = 0. \quad . \quad . \quad . \quad . \quad . \quad (14)$$

For equilibrium, the sum of the forces normal to A B must also be equal to zero; or

$$\Sigma Y = - W + \int_0^l (P - px)\,dx = 0, \quad . \quad . \quad . \quad (15)$$

from which

$$p\,l^2 = 2\,P\,l - 2\,W. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

502. Maximum Pressure. Combining (16) with (14) and reducing,

$$P = \frac{4W}{l} - \frac{6W\bar{x}}{l^2} + \frac{6M}{l^2}. \quad . \quad . \quad . \quad . \quad . \quad (17)$$

If the stability against overturning be determined algebraically, *i. e.*, by equation (12), then M and $\bar{x}$ are known, and P can be computed by equation (17).

If the wall is symmetrical $\bar{x} = \frac{1}{2} l$, and (17) becomes

$$P = \frac{W}{l} + \frac{6M}{l^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

Equation (18) is a more general form of equation (1), page 205, since in the latter there is but one external force acting, and that is horizontal.

In equation (18), notice that $\frac{W}{l}$ is the uniform pressure on $A\,B$ due to the weight of the wall; also that $\frac{6M}{l^2}$ is the increase of pressure at A due to the tendency to overturn, and that consequently the uniform pressure at B is decreased a like amount.

503. The maximum pressure may be found also in another way. Assume that N, Fig. 71, is the center of pressure. Let p_1 $(= B\,L)$ represent the pressure at B, and p_2 $(= A\,L)$ that at A; and any intermediate ordinate of the trapezoid $A\,B\,L\,K$ will represent the pressure at the corresponding point. Then, since the forces acting on $A\,B$ must be in equilibrium for translation, the area of the trapezoid will represent the entire pressure on the base $A\,B$. Stated algebraically, this is

$$\frac{p_1 + p_2}{2} l = W. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

Also, since the forces acting on $A\,B$ must be in equilibrium for rotation, the moment of the pressure to the right of N must be equal to that to the left; that is to say, the center of gravity of the trapezoid $A\,B\,L\,K$ must lie in the line $N\,J$. By the principles of analytical mechanics, the ordinate $A\,N$ to the center of gravity $A\,B\,L\,K$ is

$$\bar{x} = \frac{l}{3}\left(\frac{2p_1 + p_2}{p_2 + p_1}\right). \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

Solving (19) and (20) gives

$$p_2 = \frac{4\,W}{l} - \frac{6\,W\bar{x}}{l^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

If the wall is a right-angled triangle with the right angle at A, $\bar{x} = \frac{1}{3}\,l$, which, substituted in the above expression, shows that the pressure at A is $\frac{2\,W}{l}$, and also that the pressure at B is zero,—all of which is as it should be. *Equation* (21) *is a perfectly general expression for the pressure between any two plane surfaces pressed together by normal forces.* Notice that equation (21) is identical with the first two terms of the right-hand side of equation (17).

The form of (21) can be changed by substituting for $\bar{x}$ its value $\frac{1}{2}\,l - d$; then

$$p_2 = \frac{W}{l} + \frac{6\,W d}{l^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

Equation (22) gives the pressure at A due to the weight of the wall; but it will also give the maximum pressure on the base due to both the vertical and the horizontal forces, provided d be taken as the distance from the middle of the base to the point in which the resultant of all the forces cuts the base. Therefore we may write

$$P = \frac{W}{l} + \frac{6\,W d}{l^2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (23)$$

504. Equation (23) is the equivalent of equation (17), page 322. It is well to notice that *equation* (23) *is limited to rectangular horizontal cross-sections,* since it was assumed that the pressure on the section varies as the distance back from the toe. If the stability against overturning is determined algebraically, as by equation (12), then equation (17) is the more convenient; but if the stability is determined graphically, as in Fig. 70, then equation (22) is the simpler. Notice that if $d = \frac{1}{6}\,l$, $P = \frac{2\,W}{l}$, which is in accordance with what is known in the theory of arches as the principle of the middle third; that is, as long as the center of pressure lies within the middle third of the joint, the maximum pressure is not more than twice the mean, and there is no tension in any part of the joint.

Notice, in equation (23), that $\frac{W}{l}$ is the uniform load on the base; and also that $\frac{6\,W d}{l^2}$ is the increase of pressure due to the eccentricity of the load. It is immaterial whether the deviation d is caused by the form of the wall or by forces tending to produce overturning.

505. Tension on the Masonry. By an analysis similar to that above, it can be shown that the decrease in pressure at B, due to the overturning moment, is equal to the increase at A. If $d = \frac{1}{6}\,l$, then by equation (23) the increase at A and decrease at B is W, that is to say, the pressure at A is $2\,W$ and that at B is zero. Therefore, if the center of pressure departs more than $\frac{1}{6}\,l$ from the center of the base, there will be a minus pressure, *i. e.* tension, at B. Under this condition, the triangle $A\,V K'$, in Fig. 71, page 321, represents the total pressure, and the triangle $B\,V L'$ the total tension on the masonry,—$A\,K'$ being the maximum pressure at A, and $B\,L'$ the maximum tension at B.

If a good quality of cement mortar is used, it is not unreasonable to count upon a little resistance from tension. As a general rule, it is more economical to increase the quantity of stone than the quality of the mortar; but in dams it is necessary to use a good mortar to prevent (1) leakage, (2) disintegration on the water side, and (3) crushing. If the resistance due to tension is not included in the computation, it is an increment to the computed margin of safety.

506. If the masonry be considered as incapable of resisting by tension, then when d in equation (23) exceeds $\frac{1}{6}\,l$ the total pressure will be borne on $A\,V$, Fig. 71. In this case $A\,N'$ (the distance from A to the point where the resultant pierces the base) will be less than $\frac{1}{3}\,l$. The area of the triangle $A\,V K' = \frac{1}{2}\,A\,K' \times A\,V = \frac{1}{2}\,P \times 3\,A\,N'$. Since it is assumed that the portion $V B$ is incapable of resisting by tension, the entire weight will be borne by $A\,V$; and therefore the area of $A\,V K'$ will represent the total weight W. Hence $\frac{1}{2}\,P \times 3\,A\,N' = W$, or

$$P = \frac{2\,W}{3\,A\,N'} = \frac{2\,W}{3\,(\frac{1}{2}\,l - d)}. \quad . \quad . \quad . \quad . \quad . \qquad (24)$$

To illustrate the difference between equations (23) and (24),

assume that the distance from the resultant to the center of the base is one quarter of the length of the base, *i. e.*, assume that $d = \frac{1}{4} l$. Then, by equation (23), the maximum pressure at A is

$$P = \frac{W}{l} + \frac{6\, W l}{l^2 4} = 2\tfrac{1}{2} \frac{W}{l}, \quad \ldots \ldots \quad (25)$$

and by equation (24) it is

$$P = \frac{2\, W}{3\,(\frac{1}{2} l - \frac{1}{4} l)} = 2\tfrac{2}{3} \frac{W}{l}. \quad \ldots \ldots \quad (26)$$

That is to say, if the masonry is capable of resisting tension, equation (25) shows that the maximum pressure is $2\frac{1}{2}$ times the pressure due to the weight alone; and if the masonry is incapable of resisting tension, equation (26) shows that the maximum pressure is $2\frac{2}{3}$ times the pressure due to the weight alone.

Notice that equation (24) is not applicable when d is less than $\frac{1}{6} l$; in that case, equation (23) must be used.

507. Limiting Pressure. As a preliminary to the actual designing of the section, it is necessary to fix upon the maximum pressure per square foot to which it is proposed to subject the masonry. Of course, the allowable pressure depends upon the quality of the masonry, and also upon the conditions assumed in making the computations. It appears to be the custom, in practical computations, to neglect the vertical pressure on the inside face of the dam, *i. e.*, to assume that M_2, equation (11), page 319, is zero; this assumption is always on the safe side, and makes the maximum pressure on the outside toe appear greater than it really is. Computed in this way, the maximum pressure on rubble masonry in cement mortar in some of the great dams of the world is from 11 to 14 tons per sq. ft. The proposed Quaker Bridge dam is designed for a maximum pressure of 16.6 tons per sq. ft. on massive rubble in Portland cement mortar.

For data on the strength of stone and brick masonry, see §§ 221–23 and §§ 246–48, respectively.

508. The actual pressure at the toe will probably be less than that computed as above. It was assumed that the weight of the wall was uniformly distributed over the base; but if the batter is considerable, it is probable that the pressure due to the weight of the wall will not vary uniformly from one side of the base to the

other, but will be greater on the central portions. The actual maximum will, therefore, probably occur at some distance back from the toe. Neither the actual maximum nor the point at which it occurs can be determined.

Professor Rankine claims that the limiting pressure for the outside toe should be less than for the inside toe. Notice that the preceding method determines the maximum vertical pressure. When the maximum pressure on the inside toe occurs, the only force acting is the vertical pressure; but when the maximum on the outside occurs, the thrust of the water also is acting, and therefore the actual pressure is the resultant of the two. With the present state of our knowledge, we can not determine the effect of a horizontal component upon the vertical resistance of a block of stone, but it must weaken it somewhat.

ART. 2. OUTLINES OF THE DESIGN.

509. WIDTH ON TOP. As far as the forces already considered are concerned, the width of the wall at the top might be nothing, since at this point there is neither a pressure of water nor any weight of masonry. But in practice we must consider the shock of waves and ice, which in certain cases may acquire great force and prove very destructive to the upper portion of the dam. This force can not be computed, and hence the width on top must be assumed. This width depends to a certain extent upon the height and length of the dam. The top of large dams may be used as a roadway. Krantz * says that it is "scarcely possible to reduce the top width below 2 metres (6.5 ft.) for small ponds, nor necessary to make it more than 5 metres (16.4 ft.) for the largest."

Fig. 72, page 328, gives the width on top of Krantz's profile type, and also of the profile recommended by the engineers of the Aqueduct Commission for the proposed Quaker Bridge dam.

510. THE PROFILE. In designing the vertical cross section of a gravity dam to resist still water, it is necessary to fulfill three conditions: (1) To prevent sliding forward, equation (4), page 315, must be satisfied; (2) to resist overturning, equation (12), page 319, must be satisfied; and (3) to resist crushing, equation (23), page 323, or (24), page 324, must be satisfied. As these equations really

* "Study of Reservoir Walls," Mahan's translation, p. 35.

involve only three variables, viz.: h, b_1, and b',—the height of the dam and the batter of the two faces,—they can be satisfied exactly. It has been shown that there is no danger of the dam's sliding forward even if the width on top is zero; and hence there are practically but two conditions to be fulfilled and two variables to be determined. To prevent overturning when the reservoir is full, equation (12) must be satisfied; and to prevent crushing, equation (23)—or (24)—must be satisfied for the point A (Figs. 69, 70, etc.) when the reservoir is full, and for B when the reservoir is empty.

Although it is possible to satisfy these conditions exactly, the theoretical profile can be obtained only by successive approximations. This is done by dividing the profile into elementary horizontal layers, beginning at the top, and determining the dimension of the base of each layer separately. The theoretical width at the top being zero and the actual width being considerable, a portion of the section at the top of the dam will be rectangular. A layer being given, and the profile of the portion above it being known, certain dimensions are assumed for the lower base of the layer; and the stability against overturning is then determined by applying equation (12), or by the method of Fig. 70 (page 320). The maximum pressure at A is then found by applying equation (17) or (23), after which the maximum pressure at B when the reservoir is empty must be determined by applying equation (23). If the first dimensions do not give results in accordance with the limiting conditions, others must be assumed and the computations repeated. A third approximation will probably rarely be needed.

It is not necessary to attempt to satisfy these equations precisely, since there are a number of unknown and unknowable factors, as the weight of the stone, the quality of the mortar, the character of the foundation, the quality of the masonry, the hydrostatic pressure under the mass, the amount of elastic yielding, the force of the waves and of the ice, etc., which have more to do with the ultimate stability of a dam than the mathematically exact profile. It is therefore sufficient to assume a trial profile, being guided in this by the matters referred to in § 511 and § 512, and test it at a few points by applying the preceding equations; a few modifications to more nearly satisfy the mathematical conditions or to simplify the profile is as far as it is wise to carry the theoretical determination of the profile.

511. Krantz's Study of Reservoir Walls, translated from the French by Capt. F. A. Mahan, U. S. A., gives the theoretical profiles for dams from 16.40 ft. (5 metres) to 164 ft. (50 metres) high. The faces are arcs of circles. The mathematical work of determining the profiles is not given; but it is evident that the polygonal profile was deduced as above described, and that an arc of a circle was then drawn to average the irregularities. The largest of these profiles is shown in Fig. 72 by the broken line. The others are simply the upper portion of the largest, with the thickness and the height of the portion above the water decreased somewhat and the radius of the faces modified correspondingly.

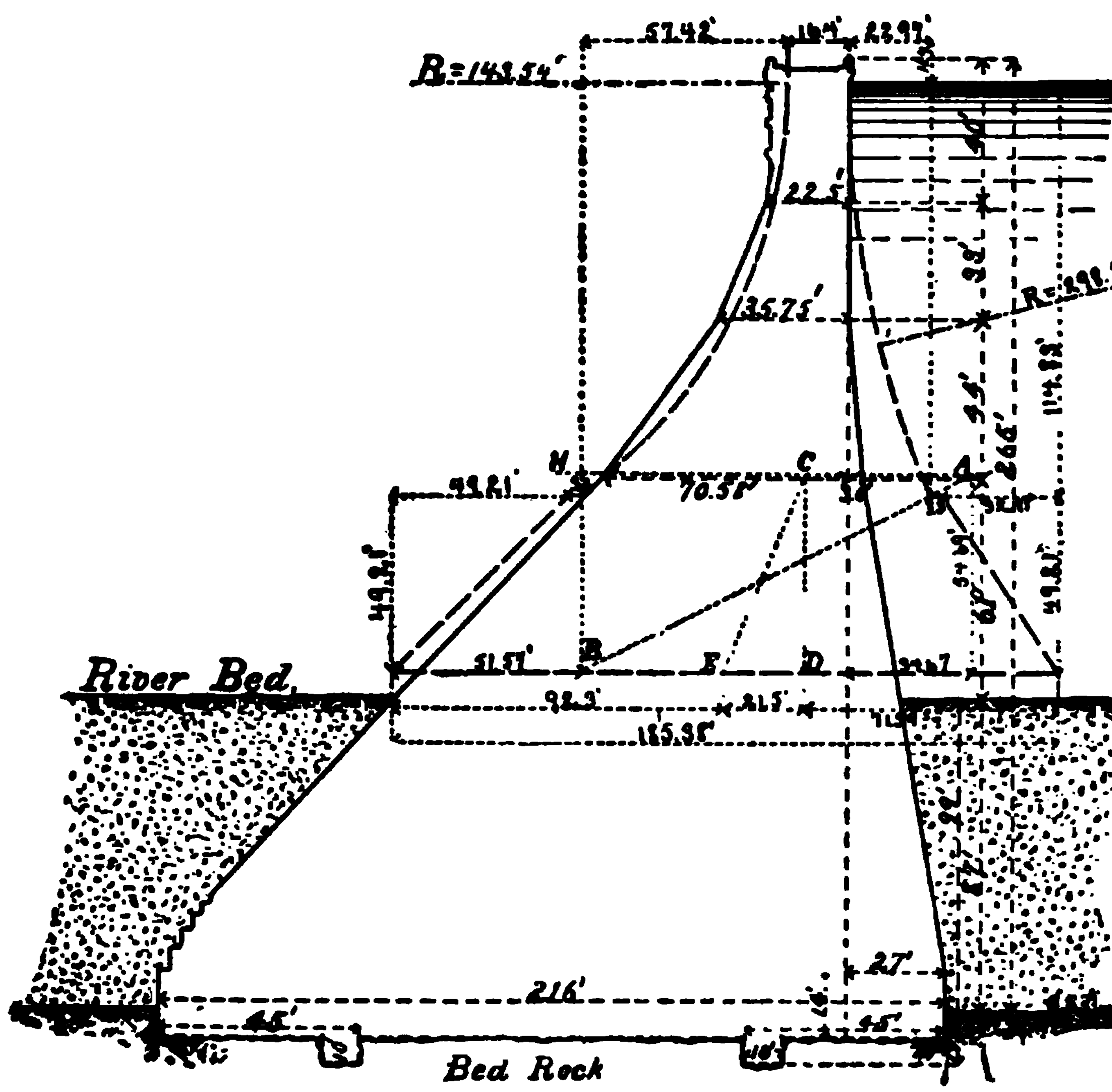

FIG. 72.

The larger profile of Fig. 72 is that recommended by the engineers of the Aqueduct Commission for the proposed Quaker Bridge dam. The profiles of most of the high masonry dams of the world

are exceedingly extravagant, and hence it is not worth while to give examples.

512. Prof. Wm. Cain has shown * that the equations of condition are nearly satisfied by a cross section composed of two trapezoids, the lower and larger of which is the lower part of a triangle having its base on the foundation of the dam and its apex at the surface of the water, and the upper trapezoid having for its top the predetermined width of the dam on top (§ 509), and for its sides nearly vertical lines which intersect the sides of the lower trapezoid. The width of the dam at the bottom is obtained by applying the equations of condition as above. The relative batter of the up-stream and down-stream faces depends upon the relative factors of safety for crushing and overturning. This section gives a factor of safety which increases from bottom to top,—an important feature.

513. THE PLAN. If the wall is to be one side of a rectangular reservoir, all the vertical sections will be alike; and therefore the heel, the toe, and the crest will all be straight. If the wall is to be a dam across a narrow valley, the height of the masonry, and consequently its thickness at the bottom, will be greater at the center than at the sides. In this case the several vertical cross sections may be placed so that (1) the crest will be straight, or (2) so that the heel will be straight in plan, or (3) so that the toe will be straight in plan. Since the up-stream face of the theoretical profile is nearly vertical (see Fig. 72), there will be very little difference in the form of the dam whether the several cross sections are placed in the first or the second position as above. If the crest is straight, the heel, in plan, will be nearly so; if the crest is straight, the toe, in plan, will be the arc of a circle such that the middle ordinate to a chord equal to the span (length of the crest) will be equal to the maximum thickness of the dam; and if the toe is made straight, the crest will become a circle of the same radius. This shows that strictly speaking it is impossible to have a straight gravity dam across a valley, since either the crest or toe must be curved. The question then arises as to the relative merits of these two forms.

514. Straight Crest vs. Straight Toe. The amount of masonry

* *Engineering News*, vol. xix. pp. 512-13.

in the two forms is the same, since the vertical sections at all points are alike in both.*

The stability of the two forms, considered only as gravity dams, is the same, since the cross sections at like distances from the center are the same.

The form with a curved crest and straight toe will have a slight advantage due to its possible action as an arch. However, it is not necessary to discuss further the relative advantages of these two types, since it will presently be shown that both the toe and the crest of a gravity dam should be curved.

515. Gravity vs. Arch Dams. A dam of the pure gravity type is one in which the sole reliance for stability is the weight of the masonry. A dam of the pure arch type is one relying solely upon the arched form for stability. With the arched dam, the pressure of the water is transmitted laterally through the horizontal sections to the abutments (side hills). The thickness of the masonry is so small that the resultant of the horizontal pressure of the water and the weight of the masonry passes outside of the toe; and hence, considered only as a gravity dam, is in a state of unstable equilibrium. If such a dam fails, it will probably be by the crushing of the masonry at the ends of the horizontal arches. In the present state of our knowledge concerning the elastic yielding of masonry, we can not determine, with any considerable degree of accuracy, the distribution of the pressure over the cross section of the arch (see Art. 1, Chap. XVIII).

If it were not for the incompleteness of our knowledge of the laws governing the stability of masonry arches, the arch dam would doubtless be the best type form, since it requires less masonry for any particular case than the pure gravity form. The best information we have in regard to the stability of masonry arches is derived from experience. The largest vertical masonry arch in the world has a span of only 220 feet. There are but two dams of the pure arch type in the world, viz.: the Zola † in France and the

* If the valley across which the dam is built has any considerable longitudinal slope, as it usually will have, there will be a slight difference according to the relative position of the two forms. If two ends remain at the same place, the straight toe throws the dam farther up the valley, makes the base higher, and consequently slightly decreases the amount of masonry.

† For description, see Report on Quaker Bridge Dam, *Engineering News*, vol. xix. p. 6 *et seq.*

Bear Valley* in Southern California. The length of the former is 205 feet on top, height 122 feet, and radius 158 feet; the length of the latter is 230 feet on top, height 64 feet, radius of top 335 feet and of the bottom 226 feet. The experience with *large* arches is so limited (see Table 63, page 502), as to render it unwise to make the stability of a dam depend wholly upon its action as an arch, except under the most favorable conditions as to rigid side-hills and also under the most unfavorable conditions as to cost of masonry. Notice that with a dam of the pure arch type, the failure of one part is liable to cause the failure of the whole; while with a gravity section, there is much less danger of this. Further, since the average pressure on the end arch stones increases with the span, the arch form is most suitable for short dams.

516. Curved Gravity Dams. Although it is not generally wise to make the stability of a dam depend entirely upon its action as an arch, a gravity dam should be built in the form of an arch, *i. e.*, with both crest and toe curved, and thus secure some of the advantages of the arch type. The vertical cross section should be so proportioned as to resist the water pressure by the weight of the masonry alone, and then any arch-like action will give an additional margin for safety. If the section is proportioned to resist by its weight alone, arch action can take place only by the elastic yielding of the masonry under the water pressure; but it is known that masonry will yield somewhat, and that therefore there will be some arch action in a curved gravity dam. Since but little is known about the elasticity of stone, brick, and mortar (see § 16), and nothing at all about the elasticity of actual masonry, it is impossible to determine the amount of arch action, *i. e.*, the amount of pressure that is transmitted laterally to the abutments (side-hills).

That it is possible for a dam to act as an arch and a gravity dam at the same time is shown as follows: "Conceive a dam of the pure arch type, of thin rectangular cross section so as to have no appreciable gravity stability. Conceive the dam to be made up of successive horizontal arches with key-stones vertically over each other. The thrust in each arch will increase with the depth, but the spans will, under the ordinary practical conditions, decrease with the depth, so that the tendency to 'settle at the crown' (move horizontally) will be approximately equal in each. If now we adopt

* For description, see *Engineering News*, vol. xx. pp. 513–15.

a triangular in place of a rectangular cross section, we increase the areas and decrease the unit pressures from arch-thrust as we go down, and hence decrease compression and consequent horizontal 'settlement' of the arches; in other words, we introduce a tendency in the water face of the dam to rotate about its lower edge. But this is precisely the tendency which results from the elastic action of the mass in respect to gravity stability, which latter we have at the same time introduced by adopting the gravity section. Hence the two act in perfect harmony, and there will be a certain size of triangular section (theoretically,—practically it could not be exact) at which precisely half the stability will be due to arch action and half to gravity action, each acting without any appreciable conflict or interference with the other." *

517. In addition to the increased stability of a curved gravity dam due to arch action, the curved form has another advantage. The pressure of the water on the back of the arch is everywhere perpendicular to the up-stream face, and can be decomposed into two components—one perpendicular to the chord (the span) of the arch, and the other parallel to the chord of the arc. The first component is resisted by the gravity and arch stability of the dam, and the second throws the entire up-stream face into compression. The aggregate of this lateral pressure is equal to the water pressure on the projection of the up-stream face on a vertical plane perpendicular to the span of the dam. This pressure has a tendency to close all vertical cracks and to consolidate the masonry transversely, —which effect is very desirable, as the vertical joints are always less perfectly filled than the horizontal ones. This pressure also prepares the dam to act as an arch earlier than it would otherwise do, and hence makes available a larger amount of stability due to arch action.

The compression due to these lateral components is entirely independent of the arch action of the dam, since the arch action would take place if the pressure on the dam were everywhere perpendicular to the chord of the arch. Further, it in no way weakens the dam, since considered as a gravity dam the effect of the thrust of the water is to relieve the pressure on the back face, and considered as an arch the maximum pressure occurs at the sides of the down-stream face.

* Editorial in *Engineering News*, vol. xix. p. 272.

The curved dam is a little longer than a straight one, and hence would cost a little more. The difference in length between a chord and its arc is given, to a close degree of approximation, by the formula

$$a = c + \frac{c^3}{24\,r^2} = c\left(1 + \frac{c^2}{24\,r^2}\right),$$

in which a = the length of the arc, c = the length of the chord, and r = the radius. This shows that the increase in length due to the arched form is comparatively slight. For example, if the chord is equal to the radius, the arch is only $\frac{1}{24}$, or 4 per cent., longer than the chord. Furthermore, the additional cost is less, proportionally, than the additional quantity of masonry; for example, 10 per cent. additional masonry will add less than 10 per cent. to the cost.

518. Of the twenty-five most important masonry dams in the world, two are of the pure arch type, sixteen are of the curved gravity type, and eight are of the straight gravity type. The eight highest dams are of the curved gravity type.*

519. **Quality of the Masonry.** It is a well settled principle that any masonry structure which sustains a vertical load should have no continuous vertical joints. Dams support both a horizontal and a vertical pressure, and hence neither the vertical nor the horizontal joints should be continuous. This requires that the masonry shall be broken ashlar (Fig. 39, page 136) or random squared-stone masonry (Fig. 44, page 137), or uncoursed rubble (Fig. 45, page 137). The last is generally employed, particularly for large dams. The joints on the faces should be as thin as possible, to diminish the effect of the weather on the mortar and also the cost of repointing. In ordinary walls much more care is given to filling completely the horizontal than the vertical ones; but in dams and reservoir walls it is important that the vertical joints also shall be completely filled.

To prevent leakage, it is very important that all spaces between the stones should be filled completely with good mortar, or better, with mortar impervious to water (see § 141). If the stone itself is not impervious, the wall may be made water tight by the application of Sylvester's washes (§ 263) to the inside face of the dam.

* For source of information concerning these dams, see § 520—Bibliography of Masonry Dams.

520. Bibliography of Masonry Dams. *Design and Construction of Masonry Dams*, Rankine, (Miscellaneous Scientific Papers, pp. 550–61.) *Study of Reservoir Walls*, Krantz, (translated from the French by Capt. F. A. Mahan, U. S. A.) *Profiles of High Masonry Dams*, McMaster, (published in Van Nostrand's Engineering Magazine and also as No. 6 of Van Nostrand's Science Series.) *Strains in High Masonry Dams*, E. Sherman Gould, (Van Nostrand's Engineering Magazine, vol. 30, p. 265 *et seq.*). *Historical and Descriptive Review of Earth and Masonry Dams, with Plans*, David Gravel, (Scientific American Supplement, No. 595 (May 28, 1887), pp. 9496–9500.) Wegmann's *Design and Construction of Masonry Dams* gives an account of methods employed in determining the profile of the proposed Quaker Bridge dam, and also contains illustrations of the high masonry dams of the world. For a general discussion of high masonry dams, including a consideration of the best form for the horizontal cross section, a full description of the proposed Quaker Bridge dam and a comparison of it with other great dams, and many valuable points concerning practical details, see numerous reports, correspondence, and editorials in *Engineering News*, January to June, 1888 (vol. 19). The above articles contain many references to the literature, mostly French, of high masonry dams.

Art. 3. Rock Fill Dams.

521. There are three well-known types of dams, which have been in use from time immemorial: earth bank, timber crib-work, and masonry. Recent engineering practice on the Pacific coast has introduced another type, viz.: the *Rock Fill Dam*, which is of too much importance to pass by without a mention here, although strictly it can not be classed as masonry construction.

A rock fill dam consists of an embankment of irregular stones thrown in loosely, except that sometimes the faces are laid by hand. If the overflow is to discharge over the crest, the largest stones should be placed on the down-stream slope. The dam may be made practically water tight (1) by filling the voids with smaller stones, gravel, sand, and earth, or (2) by placing any desired thickness of earth and puddle on the up-stream face, or (3) by covering the water slope with one or more thicknesses of planking, which is calked and sometimes also pointed. Either the first or second method

would make a dam practically water tight from the beginning, and it would grow tighter with age; the third method, if carefully executed, would make the dam absolutely water tight at the beginning, but would decay, since the upper part of the sheeting would ordinarily be alternately wet and dry.

A great number of rock fill dams have been built on the Pacific slope in the past few years, for mining and irrigating purposes. A dam of this character has recently been completed on the Hassayampa River in Arizona, of the following dimensions: "Height, 110 ft.; base, 135 ft.; top width, 10 ft.; length on top, 400 ft.; water slope, 20 ft. horizontal to 47 ft. vertical ($\frac{1}{2}$ to 1); back slopes, 70 ft. horizontal to 180 ft. vertical ($\frac{2}{5}$ to 1); contents, 46,000 cu. yds.; cost, by contract, $2.40 per cu. yd."* It is proposed to build a dam of this character in California 250 feet high, which is about 80 feet higher than any existing masonry dam, and practically is nearly the same amount higher than the proposed Quaker Bridge dam (Fig. 72, page 328).

522. "Earth dams are good and useful when only still water not running over the crest is to be dealt with. Counting reservoir walls as dams, which they are, earth dams are vastly more used than any other. They must be made with the greatest care, and, if of any considerable height, an inner wall of puddle is necessary to their integrity. They must be carried down to firm and impervious subsoil of some kind, or they are worthless. Any considerable leak is at once fatal to them; and they are also subject to serious injury from muskrats, crabs, etc. Nevertheless, many earth dams of great age and great height exist, and bid fair to exist for ages, showing that it is entirely possible to make them secure."

Stone-filled timber cribs have been very much used for dams; but such structures are sure to rot in time, since the timber can not always be kept wet. It seems probable that in most instances where cribs have been used a rock-fill dam would have been better, cheaper, and more durable.

Masonry dams of all sizes, proportions, and ages exist in great abundance, and the entire suitability of masonry for the construction of dams is well established. This class of dams is to be preferred where large quantities of stone are not near at hand, or where leakage is undesirable because of loss of water or of injury to land be-

* *Engineering News*, vol. xx. p. 232.

low, or where space is valuable, or where the surroundings require a dam of good appearance.

523. "These three types afford an adequate choice for nearly all requirements, but it is obvious that they are open to certain common objections from which the fourth type—a rock-fill dam—is free. They are all comparatively costly; they require a good deal of labor, and much of it skilled and faithful labor, for their construction; they can only with great inconvenience be constructed with water around them, which for the most part must be kept away by costly coffer-dams or diversions of channels; above all, a leak is always a source of danger, and is apt to be destructive. They are all of them, as it were, during all their existence, in unstable equilibrium—all right so long as the balance of forces remains undisturbed, and seriously endangered by a variety of causes which may disturb it. On the other hand a rock-fill dam is by the very process of its construction, if conducted with reasonable judgment, a structure which tends to improve with time, and which can not be injured but may be benefited by causes which threaten the other and more artificial types; in other words, it is a structure which may not be very tight, but which is in stable equilibrium as respects all disturbing causes, being improved and never injured by them.

"A rock-fill dam is appropriate where the bed on which it rests is either rock, hard-pan, stiff clay, or some other impervious and almost unwashable material. The bed may be more or less overlaid with gravel or loose material without harm, if it be possible to remove the loose material in advance, and if there be current enough to remove it from under the foot of the dam, as the work of construction progresses, it will not even involve extra expense or delay, and the dam may be begun on top of the stratum without apparent regard to it; but whenever there is any considerable stratum of loose material, a rock-fill dam can only be built by backing it with earth or puddle as a timber dam would be, and the necessity of providing a proper apron to receive the overflow may make a timber or crib dam the more economical. It is obvious that the place of all places for the proper use of such a rock-fill dam is where leakage is of no importance, either from the loss of water or from injury to land below; where skilled labor is scarce and costly, and simplicity of work rather than aggregate quantities

the important consideration; where good materials for masonry are scarce or absent; and where the surroundings do not demand attention to the question of appearance." *

The greatest economy in this form of dam occurs when the fill is made in water; and it is particularly advantageous in the canalization of rivers, *i. e.*, in forming pools in rivers for the benefit of navigation. It has been proposed to use rock-fill dams exclusively in the construction of the Nicaragua canal.

524. In California the cost of this class of dams varies from $2 to $3 per cubic yard, including all accessories, which is said to be about 50 per cent. cheaper than for earth dams of equal area of transverse cross section.

* Editorial in *Engineering News*, vol. xx. p. 70.

CHAPTER XIV.

RETAINING WALLS.

525. Definitions. *Retaining wall* is a wall of masonry for sustaining the pressure of earth deposited behind it after it is built.

Face wall, or *slope wall*, is a species of retaining wall built against the face of earth in its undisturbed and natural position. Obviously it is much less important and involves less difficulties than a true retaining wall.

Buttresses are projections in the front of the wall to strengthen it. They are not often used, on account of their unsightliness, except as a remedy when a wall is seen to be failing.

Counterforts are projections at the rear of the wall to increase its strength. They are of doubtful economy, and were much more frequently used formerly than now.

Land-ties are long iron rods which connect the face of the wall with a mass of masonry, a large iron plate, or a large wooden post bedded in the earth behind the wall, to give additional resistance to overturning.

Surcharge. If the material to be supported slopes up and back from the top of the wall, the earth above the top is called the *surcharge.*

Among military men, a retaining wall is called a *revetment.* When the earth is level with the top, a *scarp revetment;* when above it, a *counterscarp revetment*, or a *demi-revetment;* when the face of the wall is battered, a *sloping revetment;* when the back is battered, a *countersloping revetment.* The batter is called the *talus.*

Retaining walls are frequently employed in railroad work, on canals, about harbors, etc.; and the principles involved in their construction have more or less direct application in arches, in tunneling and mining, in timbering of shafts, and in the excavation of deep trenches for sewers, etc., and in military engineering.

526. Method of Failure. A retaining wall may fail (1) by revolving about the front of any horizontal joint, or (2) by sliding on the plane of any horizontal joint, or (3) by the bulging of the body of the masonry. The first is much the most frequent mode of

failure, and the second is the least frequent. The wall can not fail by the center's bulging out, unless some force acts to keep the top from moving forward,—as in a cellar wall, the abutments of arches, etc.

527. **Difficulties.** In the discussion of the stability of dams, it was shown that in order to completely determine the effect of the thrust of the water against the wall, it is necessary to know (1) the amount of the pressure, (2) its point of application, and (3) the direction of its line of action. Similarly, to determine the effect of the thrust of a bank of earth against a wall, it is necessary to know (1) the amount of the pressure, (2) its point of application, and (3) its line of action. The determination of these three quantities requires three equations. The resistance of the wall both to sliding and to overturning can be found with sufficient accuracy, as has already been explained in Chapter XIII—Dams;—but the other elements of the problem are, in the present state of our knowledge, indeterminate.

The origin of the difficulties may be explained briefly as follows. *A B* represents a retaining wall; *A D* is the surface of the ground. The earth has a tendency to break away and come down some line as *C D*. The force tending to bring the earth down is its weight; the forces tending to keep it from coming down are the friction and cohesion along the line *C D*. The pressure against the wall depends upon the form of the line *C D*. If the constants of weight, friction, and cohesion of any particular ground were known, the form of *C D* and also the amount of the thrust on the wall could be determined.

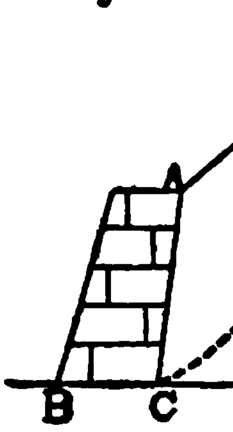

Fig. 73.

Notwithstanding the fact that since the earliest ages constructors have known by practical experience that a mass of earthwork will exert a severe lateral pressure upon a wall or other retaining structure, there is probably no other subject connected with the constructor's art in which there exists the same lack of exact experimental data. This lack is doubtless due, in part at least, to a reliance upon theoretical investigations. Of course, mathematical investigations unsupported by experiments or experience are a very uncertain guide.

This subject will be discussed further under the heads (1) Theoretical Formulas, and (2) Empirical Rules.

ART. 1. THEORETICAL FORMULAS.

528. A great variety of theories have been presented, but all rest upon an uncertain foundation of assumption, and all are more or less defective and self-contradictory. The theories of the stability of retaining walls in most frequent use will now be stated, and the underlying assumptions and the defects of each will be pointed out.

529. FIRST ASSUMPTION. All theories assume that the surface of rupture, $C\,D$, Fig. 73, is a plane. This is equivalent to assuming that the soil is devoid of cohesion, and is inelastic and homogeneous, and also that if a mass of such material be sustained by a wall, there is a certain plane, called the plane of rupture, along which the particles are in equilibrium, *i. e.*, are just on the point of moving. This assumption would be nearly correct in the case of clean, sharp sand, but would be considerably in error with a tough, tenacious soil.

This assumption gives the data by which the amount of the thrust of the earth can be computed; that is to say, this assumption furnishes the conditions from which one of the equations may be established.

530. SECOND ASSUMPTION. A second assumption which is always made is that the point of application of the lateral pressure of the earth is one third of the height of the wall from the bottom. The total pressure on the wall varies as some function of the height; and it is assumed to vary as the square of the height, and that therefore the center of pressure is at a point two thirds of the depth below the top. This is equivalent to assuming that the variation of the pressure in a mass of earth is the same as in a liquid, *i. e.*, that the material is devoid of internal friction.

This assumption furnishes the second of the equations required to determine the effect of the thrust of earth against a retaining wall.

531. THIRD ASSUMPTION. The third equation is obtained by assuming the direction of the pressure. There are different theories based on different assumptions as to this direction. The theories also differ because of the difference in the methods of deducing an equation from the second assumption. The principal theories will now be considered briefly.

532. COULOMB'S THEORY. The theory advanced by Coulomb in 1784 was the first to even approximate the actual conditions, and his method is the basis of nearly all formulas used by engineers at the present time. It has been taken up and followed out to its consequences by Prony (1802), Mayniel (1808), Française (1820), Navier (1826), Audoy and Poncelet (1840), Hagen (1853), Scheffler (1857), and Moseley, as well as a host of others, in recent times.

Coulomb *assumed* (1) that the line $D\ C$, Fig. 73 (page 339), is a straight line, down which the prism $A\ C\ D$ tends to slide; (2) that the resultant pressure is applied at a point two thirds of the depth below the top; and (3) that the pressure exerted by this mass on the wall is normal to its back face, which is equivalent to neglecting the friction of the earth against the back of the wall. He decomposed the weight, W, of the prism $A\ C\ D$, Fig. 74, and the reaction, R, of the wall into two components respectively, parallel and perpendicular to the surface of rupture, $D\ C$. The difference of these parallel components, $P_1 - P_2$, he placed equal to the prism's resistance to sliding; and assumed the latter to be equal to $\mu\ N_1$, in which

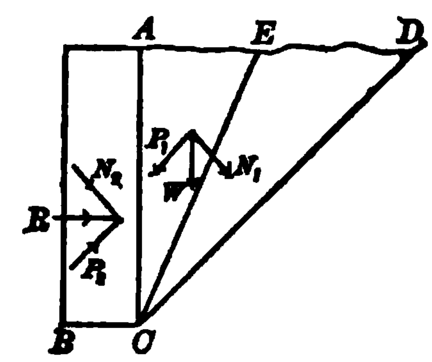

FIG. 74.

μ is the co-efficient of friction. There is some prism, $A\ C\ D$, the pressure of which against the wall is just sufficient to cause *sliding*. The amount of this pressure will depend upon the weight, w, of a unit of volume of the backing; upon the height, h, of the wall; upon the co-efficient of friction, μ, of earth on earth; and upon the distance $A\ D$, which call x.

Under the conditions assumed, it is possible to state a value of R in terms of h, w, μ, and x. Coulomb assumed R to vary as x, and differentiated the value of R to find the position of the surface of rupture, $D\ C$, for a maximum pressure on the wall. This leads to the simple conclusion that the lateral pressure exerted by a bank of earth with a *horizontal top* is simply that due to the wedge-shaped mass included between the vertical back of the wall and a line bisecting the angle between the vertical and the slope of repose of the material;* that is, the pressure of the earth against the wall $A\ B$,

* For an algebraic demonstration, see Moseley's Mechanics of Engineering (2d Amer. Ed.), pp. 413-16; for a graphical demonstration, see Van Nostrand's Engineering Magazine, vol. ix. p. 202, and vol. xxii. p. 267.

Fig. 74, is equal to the pressure of the prism $A\ CE$ sliding along a perfectly smooth plane $C\ E$, which bisects the angle of repose, $A\ C\ D$.

No satisfactory proof has been given of the correctness of this procedure by either Coulomb or any one else; and no defense has ever been made against a number of serious objections to it which have been raised. Experiments show that the lateral pressure of the prism $A\ B\ C$, Fig. 75, between two boards $A\ B$ and $A\ C$, against $A\ B$, "is quite as much when the board $A\ C$ is at the slope of repose, $1\frac{1}{2}$ to 1, as when it is at half the angle; and there was hardly any difference whether the board was horizontal, or at a slope of $\frac{1}{4}$ to 1, or at any intermediate slope." *

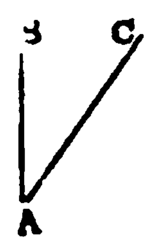

Fig. 75.

533. By this theory the pressure of the wedge $A\ C\ D$ (Fig. 74) is

$$P = \tfrac{1}{2}\, w\, h^2 \tan^2 \tfrac{1}{2} ACD, \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which w is the weight of a unit of the material to be supported, and h is the height of the wall. This thrust is assumed to act two thirds of $A\ C$, Fig. 74, below A. Or, in other words, the thrust of the prism is equivalent to the pressure of a liquid whose weight per unit of volume is $w \tan^2 \frac{1}{2}\ ACD$.

Equating the moment of the overturning force and the moments of resistance in terms of the unknown thickness, and solving the equation, gives the thickness which the wall must have to be on the point of overturning. For example, assume that it is desired to determine the thickness, t, of a vertical rectangular wall. Represent the weight of a cubic foot of the masonry by W. Then placing the moment of the wall equal to the amount of the thrust of the earth, gives

$$W\, h\, t \,.\, \tfrac{1}{2}\, t = P \,.\, \tfrac{1}{3}\, h. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Solving equations (1) and (2) gives

$$t = h \tan \tfrac{1}{2} ACD \sqrt{\frac{w}{3\,W}}. \quad . \quad . \quad . \quad . \quad . \quad (3)$$

* Benj. Baker, an eminent English engineer, in a very interesting and instructive article on "The Actual Lateral Pressure of Earthwork," reprinted in Van Nostrand's Engineering Magazine, vol. xxv. pp. 333–42, 353–71, and 492–505, from Proc. of the Inst. of C. E.

Numerous tables have been computed which give, to a great number of decimal places, the thickness of a rectangular wall in terms of its height, the arguments being the ratio of the weights of a unit of volume of the wall and backing, and the angle of repose. Such tables are of but little practical value, as will appear presently.

534. Surcharged Walls. The rule that the plane of rupture bisects the angle between the natural slope of the earth and the back of the wall, holds good only when the top surface of the bank is horizontal and the back of the wall vertical. The formula for a surcharged wall, or for the case in which the back is not vertical, or for both combined, may be deduced * in the same general way as above; but the results for each case are too complicated for ordinary use, and each is subject to the same errors as the formula for a vertical wall and level top surface. There are a number of exceedingly ingenious graphical solutions of the resulting equations. †

535. Reliability of Coulomb's Theory. It is generally conceded that the results obtained by this method have but little practical value. "Experiments and practical experience show that walls, which according to this theory are on the point of overturning, possess on the average a factor of safety of about *two.*" ‡ One of the author's students experimented with fine shot, which appear to fulfill the fundamental assumptions of this theory, and found that the observed resistance was 1.97 times that computed by Coulomb's formula.§ The uncertainties of the fundamental assumptions and the questionableness of some of the mathematical processes are sufficient explanation of the difference between the theory and practice.

536. WEYRAUCH'S THEORY. This is the latest one, having being proposed in 1878. It was first brought to the attention of American engineers by Professor J. A. Du Bois's translations of Winkler's "Neue Theorie des Erddruckes," and Weyrauch's paper on retaining walls published in "Zeitschrift für Baukunde," 1878, Band i. Heft 2, which translation was published in the Journal of the Frank-

* See Moseley's Mechanics of Engineering, pp. 424–26.

† See Van Nostrand's Engineering Magazine, vol. ix. p. 204; and do., vol. xxv. p. 355. For references to elaborate graphical treatises on retaining walls, see Du Bois's Graphical Statics, pp. lv–lvi of Introduction.

‡ Benj. Baker in "The Actual Lateral Pressure of Earthwork." See foot-note on page 342.

§ See M. Fargusson's Bachelor's Thesis, University of Illinois.

lin Institute, vol. cviii. pp. 361–87. The following presentation of this theory is drawn mainly from that article.

This theory assumes (1) that the surface of rupture is a plane, (2) that the point of application of the resultant of the lateral pressure of the earth is at a point one third of the height of the wall from the bottom, and (3) that there is no friction between the earth and the back of the wall. It is claimed that these three are the only assumptions involved in this theory, and that the direction of the resultant pressure is deduced from the fundamental relations necessary for equilibrium under the conditions assumed.

The analysis to establish the equations for the amount and direction of the thrust of the earth is too long and too complicated to be attempted here; consequently, only the final equations will be given.

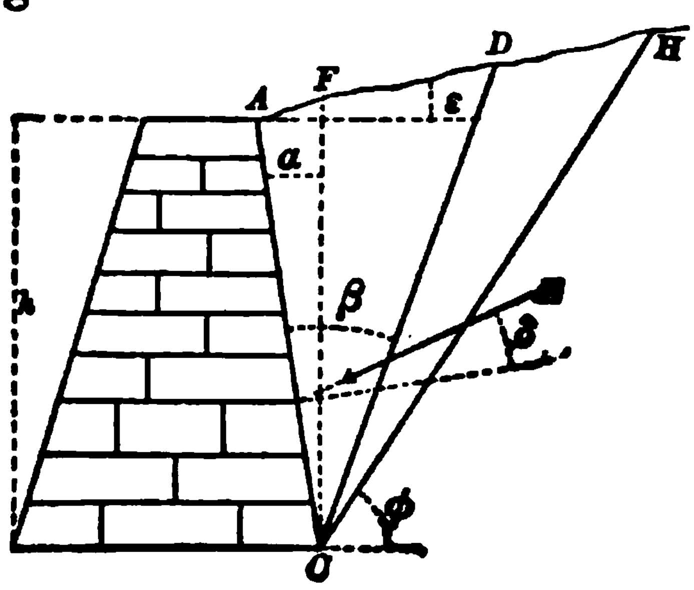

FIG. 76.

Let E = the thrust of earth against the wall.

w = the weight of a unit of the earth.

h = the height of the wall.

α = the angle the back of wall makes with the vertical.

δ = the angle which E makes with the normal to the back of the wall.

ϵ = the angle of the upper surface with the horizontal.

β = the angle of the plane of rupture with the vertical.

ϕ = the angle of repose with the horizontal.

537. General Formulas. For a plane earth-surface, horizontal or sloping up at any angle, and the back of the wall vertical or leaning forward at any angle, the general relations are *

$$E = \left[\frac{\cos(\phi - \alpha)}{(n+1)\cos\alpha}\right]^2 \frac{h^2 w}{2\cos(\alpha + \delta)}, \quad . \quad . \quad . \quad . \quad (4)$$

in which

$$n = \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \epsilon)}{\cos(\alpha + \delta)\cos(\alpha - \epsilon)}}, \quad . \quad . \quad . \quad . \quad . \quad (5)$$

* See Howe's Retaining Walls for Earth, pp. 46, 47; and also Van Nostrand's Engineering Magazine, vol. xxii. pp. 265–77.

The value of δ required in (5) can be deduced from

$$\tan\delta = \frac{\sin(2\alpha-\epsilon) - K\sin 2(\alpha-\epsilon)}{K - \cos(2\alpha-\epsilon) + K\cos 2(\alpha-\epsilon)}, \quad . \quad . \quad (6)$$

in which

$$K = \frac{\cos\epsilon - \sqrt{\cos^2\epsilon - \cos^2\phi}}{\cos^2\phi}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

538. Horizontal Earth-surface. If the upper surface of the earth is horizontal, then $\epsilon = 0$, and

$$E = \frac{\tan\alpha}{\sin(\alpha+\delta)} \cdot \frac{h^2 w}{2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

and δ can be found from

$$\tan\delta = \frac{\sin\phi\sin 2\alpha}{1 - \sin\phi\cos 2\alpha}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

If the back of the wall is vertical, $\alpha = 0$; and equation (9) gives $\delta = 0$. Therefore

$$E = \tan^2\left(45^\circ - \frac{\phi}{2}\right)\frac{h^2 w}{2}.* \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

539. Surcharge at the Natural Slope. If the upper surface of earth has the natural slope, $\epsilon = \phi$; and therefore

$$E = \left[\frac{\cos(\phi-\alpha)}{\cos\alpha}\right]^2 \frac{h^2 w}{2\cos(\alpha+\delta)}, \quad . \quad . \quad . \quad . \quad (11)$$

and δ is determined from

$$\tan\delta = \frac{\sin\phi\cos(\phi - 2\alpha)}{1 - \sin\phi\sin(\phi - 2\alpha)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

If the back of the wall is vertical, $\alpha = 0$, and $\delta = \phi$, which shows that E acts parallel to the top surface of the earth. In this case

$$E = \tfrac{1}{2}\cos\phi\ h^2 w. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

* Compare with equation (1), page 342.

540. The general equations for Weyrauch's theory, viz., equations (4), (5), (6), and (7), have not been solved for any special case, except for $\epsilon = 0$, and $\epsilon = \phi$. The reduction is very long and tedious.

541. The formulas for each of the above cases may be solved graphically,* but the explanations are too long to be given here.

542. Reliability of Weyrauch's Theory. On behalf of this theory it is claimed † that the only errors in it are those due to the neglect of the cohesion of the backing, and to assuming that the surface of rupture is a plane; and also that "it is free from all the objections which may be urged against all others, and can be used with confidence." Prof. Cain ‡ proves that Weyrauch's theory differs from Coulomb's only in the form of the results and in the manner of deducing them, and claims that both are substantially correct.

On the other hand, Weyrauch's theory is unquestionably subject to any errors which may be involved in the assumptions that the surface of rupture is a plane (see § 529), and that the point of application of the resultant pressure of the earth is at two thirds of the height of the wall from the top (see § 530). Second, the analysis purports to be perfectly general; § but it is evidently inapplicable to a wall inclined toward the earth to be supported, since the formulas make the thrust of the earth increase with the backward inclination of the wall. In fact the theory makes no difference between a wall leaning forward and one leaning backward. For a wall inclining at the angle of repose, it gives a very great lateral pressure—(see eqs. (8) and (9). Third, the mathematical process of determining the position of the surface of rupture is at least questionable. Fourth, the theory errs on the safe side, because it neglects a vertical component of the earth pressure which is independent of friction.‖

* See Jour. Frank. Inst., vol. cviii. pp. 380–85; Van Nostrand's Engineering Magazine, vol. xxii. pp. 266–73; Howe's Retaining Walls for Earth, pp. 7-12.

† By its author, Prof. Weyrauch, and also by the translator, Prof. Du Bois,—see Jour. Frank. Inst., vol. cviii. pp. 486–87.

‡ Van Nostrand's Engineering Magazine, vol. xxii. pp. 265–77.

§ See Jour. Frank. Inst., vol. cviii. p. 377; and also Howe's Retaining Walls for Earth, p. 2.

‖ In proof that such a component exists, see experiments by Siegler in *Annales des Ponts et Chaussees*, reprinted in *Scientific American Supplement*, vol. xxiv. pp. 9724–25.

543. Weyrauch's method of deducing the direction of the earth pressure assumes that there is no friction between the earth and the back of the wall, or, in other words, that the angle, δ, which the thrust of the earth makes with the back of the wall, does not depend upon the structure of the wall for its value. The formula in this form fails to agree with ordinary experience; and hence it has been proposed * to modify the general formula by considering that the angle between the resultant pressure of the earth and the back of the wall is never less than the angle of friction between the earth and the wall. The method of doing this is as follows:

If ϕ' represents the co-efficient of friction between the earth and the wall, then the direction of E must make an angle with the normal to the back face of the wall equal at least to ϕ'. To introduce ϕ' into Professor Weyrauch's theory, it is necessary to find the value of δ as given by his formula, and see if it is greater or less than ϕ'. If it is less, use the value of ϕ' to determine the direction of E; if greater, use the value of δ and omit ϕ' altogether. The value of ϕ' can not be determined accurately; but unless the back of the wall is exceedingly smooth, ϕ' will be greater than ϕ. If the back of the wall is rough rubble (§ 213) or is stepped, ϕ' will be considerably larger than ϕ. If the friction between the earth and the wall be neglected, *i. e.*, if it is assumed that $\phi' = 0$, then when rough rubble retaining walls are proportioned according to Weyrauch's theory, they will have a factor of safety considerably larger than appears from the computations.

This modification is inconsistent with the general theory, since the fundamental equations were established for that value of δ which would produce equilibrium, and the corresponding value of the thrust was deduced accordingly. It is certainly incorrect to use one direction in determining the value of the thrust and another in applying it. Further, it is not reasonable to believe that the thrust ever makes an angle with the normal to the back of the wall greater than the angle of friction, since one of the fundamental conditions of statics is that if the resultant pressure makes an angle with the normal greater than the angle of repose, motion takes place. This modification of Weyrauch's theory purports to give the relations for a state of equilibrium, and yet violates the fundamental condition necessary for equilibrium.

* By Prof. Cain in Van Nostrand's Engineering Magazine, vol. xxv. p. 92.

544. Rankine's Theory. There is another class of theories, which, in addition to the assumptions of § 530 and § 531, assume that the thrust of the earth makes an angle with the back of the wall equal to the angle of repose of the earth. Different writers arrive at their results in different ways, but most of them proceed from a consideration of the conditions of equilibrium of the earth particles, and arrive at their results by integration. Of the formulas deduced in the latter way, Rankine's * are the best known. All the theories of this class have essentially the same limitations and defects as Coulomb's and Weyrauch's.

545. Applicability of Theoretical Formulas. It is generally conceded that the ordinary theories—Coulomb's, Weyrauch's, and Rankine's,—types of the only ones for which there is any considerable show of reasonableness,—are safe; but "to assume upon theoretical grounds a lateral thrust which practice shows to be excessive, and then compensate for it by giving no factor of safety to the wall, although the common way, is not a scientific mode of procedure." This is only another reason for the statement, already made, that theoretical investigations are of but little value in designing retaining walls. The problem of the retaining wall is not one that admits of an exact mathematical solution; the conditions can not be expressed in algebraic formulas. Something must be assumed in any event, and it is far more simple and direct to assume the thickness of the wall at once than to derive the latter from equations based upon a number of uncertain assumptions.

Bear in mind that none of the above formulas apply if the back of the wall inclines towards the earth to be supported, or if the wall has a curved profile, or if the upper surface is irregular. It seems to be conceded that in these cases the surface of rupture is not a plane, and hence no theory yet proposed will apply.

In this connection it seems necessary to warn the student that not all theories for retaining walls are as nearly correct as those referred to above. Some of them, although having all the prestige of antiquity and offering the advantages of extended tables for their application, are totally valueless, being based upon unwarranted assumptions, and violating the fundamental principles of mechanics.

546. Theoretical investigations of many engineering problems which in every-day practice need not be solved with extreme accu-

* Civil Engineering, pp. 401–07.

racy, are useful in determining the relations of the various elements involved, and thus serve as a skeleton about which to group the results of experience; but the preceding discussion shows that the present theories of the stability of retaining walls are not sufficiently exact to serve even as a guide for future investigations. Furthermore, the stability of a retaining wall is not a purely mathematical problem. Often the wall is designed and built before the nature of the backing is known; and the variation of the backing, due to rain, frost, shock, extraneous loads, etc., can not be included in any formula.

ART. 2. EMPIRICAL RULES.

547. ENGLISH RULES. The eminent English engineer Benjamin Baker, who has had large experience in this line in the construction of the underground railroads of London, says, "Experience has shown that a wall [to sustain earth having a level top surface], whose thickness is one fourth of its height, and which batters 1 or 2 inches per foot on the face, possesses sufficient stability when the backing and foundation are both favorable. This allows a factor of safety of about two to cover contingencies. It has also been proved by experience that under no ordinary conditions of surcharge or heavy backing is it necessary to make a retaining wall on a solid foundation more than double the above, or one half of the height in thickness. Within these limits the engineer must vary the strength according to the conditions affecting the particular case. Outside of these limits, the structure ceases to be a retaining wall in the ordinary acceptation of the term. As a result of his own experience, the author [Benj. Baker] makes the thickness of retaining walls in ground of an average character equal to one third of the height from the top of the footings.

"'Hundreds of revetments have been built by royal engineer officers in accordance with Gen. Fanshawe's rule of some fifty years ago, which was to make the thickness of a rectangular brick wall, retaining ordinary material, 24 per cent. of the height for a batter of $\frac{1}{5}$, 25 per cent. for $\frac{1}{6}$, 26 per cent. for $\frac{1}{8}$, 27 per cent. for $\frac{1}{10}$, 28 per cent. for $\frac{1}{12}$, 30 per cent. for $\frac{1}{24}$, and 32 per cent. for a vertical wall.'" *

548. TRAUTWINE'S RULE. Trautwine † recommends that "the

* Van Nostrand's Engineering Magazine, vol. xxv. p. 370, from Proc. Inst. of C. E.

† Engineer's Pocket-Book (Ed. 1885), p. 683.

thickness of the top of the footing course of a vertical or nearly vertical wall which is to sustain a backing of sand, gravel, or earth, level top surface, when the backing is deposited loosely (as when dumped from cars, carts, etc.), for railroad practice, should not be less than the following:

Wall of cut-stone, or of first-class large-ranged rubble in mortar,	35 per cent.
" " good common scabbled mortar-rubble, or brick........	40 per cent.
" " well scabbled dry rubble............................	50 per cent.

When the backing is somewhat consolidated in horizontal layers, each of these thicknesses may be reduced; but no rule can be given for this. Since sand or gravel has no cohesion, the full dimensions as above should be used, even though the backing be deposited in layers. A mixture of sand, or earth with pebbles, paving stones, bowlders, etc., will exert a greater pressure against the wall than the materials ordinarily used for backing; and hence when such backing has to be used, the above thicknesses should be increased, say, about $\frac{1}{8}$ to $\frac{1}{6}$ part."

549. Details of Construction. The arrangement of the foundation of a retaining wall is an important matter, but has already been sufficiently discussed (see Part III, and also §§ 491 and 551). It is universally admitted that a large majority—by some put at nine out of ten, and by others at ninety-nine out of a hundred—of failures of retaining walls are due to defects in the foundation.

Retaining walls are constructed of ashlar or brick, or of either ashlar or brick backed with rubble, or of rubble either with mortar or dry. As the pressure at each bed-joint is concentrated towards the face of the wall, the larger and most regular stones should be placed on the front. Occasional stones or even courses should project beyond the back of the wall, so that the backing can rest upon them, thus increasing the resistance of the wall to overturning. This object is also promoted by building the back in steps. The coping should consist of large flat stones extending clear across the wall.

As a rule, the greatest thrust comes against retaining walls when the mortar is green and least able to resist it, which is a reason for preferring cement to lime mortar. If the backing is to be filled in before the mortar hardens, it should be deposited in thin, horizontal layers, or the wall should be supported temporarily by shores.

550. Drainage. Next to a faulty foundation, water behind the

wall is the most frequent cause of the failure of retaining walls. The water not only adds to the weight of the backing material, but also softens the material and changes the angle of repose so as to greatly increase its lateral thrust. With clayey soil, or any material resting upon a stratum of clay, this action becomes of the greatest importance. To guard against the possibility of the backing's becoming saturated with water, holes, called weepers, are left through the wall. One weep-hole, three or four inches wide and the depth of a course of masonry, is generally sufficient for every three or four square yards of front of the wall. When the backing is clean sand, the weep-holes will allow all the water to escape; but if the backing is retentive of water, a vertical layer of stones or coarse gravel should be placed next to the wall to act as a drain. An ordinary drain at the back of the wall is often useful.

When the backing is liable to be reduced to quicksand or mud by saturation with water, and when this liability can not be removed by efficient drainage, one way of making provision to resist the additional pressure which may arise from such saturation is to calculate the requisite thickness of wall as if the earth were a fluid. A puddle-wall is sometimes built against the back of dock-walls to keep out the water.

The resistance of the wall to sliding is materially increased by laying the lower courses of masonry with an inclination inward. An objection to inclining the joints, particularly in dry masonry, is that the water will enter them and be carried to the backing. This objection is sometimes met by building the face with horizontal courses, and inclining the courses in the back of the wall. The back of the wall for 2 or 3 feet from the top should have a batter of at least 1 inch in 1 foot, in order that the frost may lift the earth and not break the joints of the masonry.

Walls are sometimes built with both faces inclined toward the material to be supported, and sometimes with a curved profile; but it is generally considered unwise to do either, owing to the extra expense and trouble in construction.

551. Land Ties. Retaining walls may have their stability increased by being tied or anchored by iron rods to vertical plates of iron or blocks of stones imbedded in a firm stratum of earth at a distance behind the wall. "The holding power, per foot of breadth, of a rectangular vertical anchoring plate, the depth of whose upper

and lower edges below the surface are respectively x_1 and x_2, may be approximately calculated from the following formula:

$$H = w\frac{x_2^2 - x_1^2}{2}\,\frac{4\sin\phi}{\cos 2\phi}, \quad . \quad . \quad . \quad . \quad . \quad (14)$$

in which H is the holding-power of the plate in pounds per foot of breadth, w is the weight in pounds of a cubic foot of the earth, and ϕ its angle of repose. The center of pressure of the plate is about two thirds of its height below its upper edge,—at which point the tie-rod should be attached.

"If the retaining wall depends on the tie-rods alone for its security against sliding forward, they should be fastened to plates on the face of the wall in the line of the resultant pressure of the earth behind the wall, that is, at one third [see § 530] of the height of the wall above its base. But if the resistance to sliding forward is to be distributed between the foundation and the tie-rods, the latter should be placed at a higher level. For example, if half the horizontal thrust is to be borne by the foundation and half by the tie-rods, the latter should be fixed to the wall at two thirds of its height above the base."*

552. Relieving Arches. In extreme cases, the pressure of the earth may be sustained by relieving-arches. These consist of a row of arches having their axes and the faces of their piers at right angles to the face of a bank of earth. There may be either a single row of them or several tiers; and their front ends may be closed by a vertical wall,—which then presents the appearance of a retaining wall, although the length of the archways is such as to prevent the earth from abutting against it. Fig. 77 represents a vertical transverse section of such a wall, with two tiers of relieving arches behind it.

FIG. 77.

To determine the conditions of stability of such a structure as a whole, the horizontal pressure against the vertical plane OD may be determined, and compounded with the weight of the combined mass of masonry and earth $OAED$, to find the resultant pressure on the foundation.

* Rankine's Civil Engineering, p. 411.

CHAPTER XV.

BRIDGE ABUTMENTS.

553. General Forms. There are four forms of abutments in more or less general use. 1. A plain wall parallel to the current, shown in elevation at Fig. 78, with or without the wings $A\ D\ F$ and $B\ E\ G$. The slopes may be finished with an inclined coping, as $A\ D$, or offset at each course, as $B\ E$—usually the latter. This form may appropriately be called the *straight abutment*. 2. The wings may be swung around into the bank at any angle, as shown (in plan) in Fig. 79. The angle ϕ is usually about 30°. This form is known

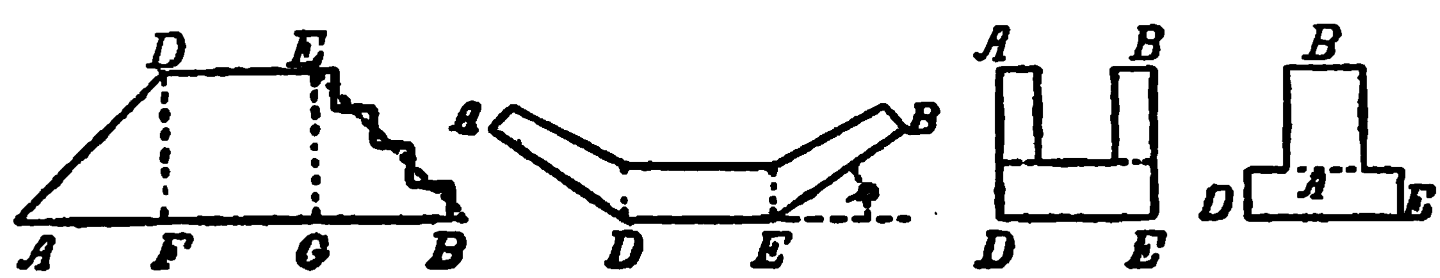

Fig. 78. Fig. 79. Fig. 80. Fig. 81.

as the *wing abutment*. 3. When ϕ of Fig. 79 becomes 90°, we have Fig. 80, which is called the U *abutment*. 4. If the wings of Fig. 80 are moved to the center of the head-wall, we get Fig. 81, which is known as the T *abutment*.

The abutment of an ordinary bridge has two offices to perform, viz., (1) to support one end of the bridge, and (2) to keep the earth embankment from sliding into the water. In Fig. 78, the portion $D\ E\ G\ F$ serves both these purposes, while the wings $A\ D\ F$ and $B\ E\ G$ act only as retaining walls. In Figs. 79 and 80, the portion $D\ E$ performs both offices, while the wings $A\ D$ and $B\ E$ are merely retaining walls. In Fig. 81 the "head" $D\ E$ supports the bridge, and the "tail," or "stem," $A\ B$ carries the train; hence the whole structure acts as a retaining wall and also supports the load. The abutment proper may fail (1) by sliding forward, (2) by bulging, or (3) by crushing; however, it is improbable that it will fail by sliding forward. Its dimensions are to be determined as for a retaining wall (Chap. XIV); but the mathematical theory of the lateral

pressure of earth is a much less perfect guide for designing bridge abutments than it is for simple retaining walls. The weight of the bridge helps the abutment to resist the thrust of the earth; but, on the other hand, the weight of the train on the embankment increases the lateral pressure against the abutment.

554. The form of the abutment to be adopted for any particular case will depend upon the locality,—whether the banks are low and flat, or steep and rocky; whether the current is swift or slow; and also upon the relative cost of earthwork and masonry. If the shore is flat, and not liable to be cut away by the current, an abutment like Fig. 78 will be sufficient and most economical. However, this form is seldom used, owing to the danger of the water's flowing along immediately behind the wall.

The form of Fig. 79 may be adopted when there is a contraction of the waterway at the bridge site, since deflecting the wing walls, above and below, slightly increases the amount of water that can pass. This advantage can be obtained, to some degree, with the straight abutment (Fig. 78) by thinning the wings on the front and leaving the back of the wings and abutments in one straight line. There is not only no hydraulic advantage, but there is a positive disadvantage, in increasing the deflection of the wings beyond, say, 10° or 15°. The more the wing departs from the face line as it swings round into the embankment, the greater its length and also the greater is the thrust upon it. The wings are not usually extended to the toe, *B*, of the embankment slope, but stop at a height, depending upon the angle of deflection and the slope, such that the earth flowing around the end of the wall will not get into the channel of the stream. It can be shown mathematically that, if the toe of the earth which flows around the end of the wing is to be kept three or four feet back from the straight line through the face of the abutment, an angle of 25° to 35° is best for economy of the material in the wing walls. This angle varies slightly with the proportions adopted for the wing wall and with the details of the masonry. This form of construction is objectionable, since the foot of the slope in front of the wing is liable to be washed away; but this could be remedied somewhat by riprapping the slope, or, better, by making the wings longer.

Fig. 78 is one extreme of Fig. 79, and Fig. 80 is the other. As the wing swings back into the embankment the thrust upon it in-

creases, reaching its maximum at an angle of about 45°; when the wing is thrown farther back the outward thrust decreases, owing to the filling up of the slope in front of the wing. Bringing the wings perpendicular to the face of the abutment, as in Fig. 80, also decreases the lateral pressure of the earth, owing to the intersection of the surfaces of rupture for the two sides, which is equivalent to removing part of the "prism of maximum thrust." If the banks of the stream are steep, the base of the wing walls of Fig. 80 may be stepped to fit the ground, thereby saving masonry. Under these conditions, also the wing abutment, Fig. 79, can be treated in the same way; but the saving is considerably less. When the masonry is stepped off in this way, the angle thus formed becomes the weakest part of the masonry; but, as the masonry has a large excess of strength, there is not much probability of danger from this cause, provided the work is executed with reasonable care.

555. Fig. 81 is the most common form of abutment. For equal amounts of masonry, wing abutments give better protection to the embankments than T abutments. The latter are more stable, because the center of gravity of the masonry is farther back from the line of the face of the abutment, about which line the abutment must turn or along which it will first crush. The amount of masonry in tall T abutments can be decreased by building the tail wall hollow, or by introducing arches under it. The more massive the masonry, the cheaper it can be constructed; and, for this reason, it is probable that the simple T abutment is cheaper than the U abutment, although the latter may have less masonry in it. On the other hand, the opportunities for inspecting the masonry during construction are better with the U than with the T abutment, and hence the former is usually better built than the latter. This is an important item, since it is somewhat common for railroad masonry to fail by being shaken to pieces by the passage of trains.

556. Wing Abutment. Fig. 82 shows a common form of the wing abutment. This one is finished with stone pedestal blocks—marked *B* in plan, *A* in elevation, and *C* in section,—which is not always done. The thickness of pedestal blocks and the thickness of the coping under the pedestal blocks vary slightly with the span (see § 558). The height of the parapet wall, or dirt wall (the wall which keeps back the top of the embankment, marked *P W* in section), will vary with the style of the bridge, but should not have

a thickness less than four tenths of its height (see §§ 547 and 548). The bridge often rests directly upon the coping. The top dimensions of the abutment will depend somewhat upon the size and form of bridge; but for railroad bridges it will usually not be less than 5 ft. wide by 20 ft. long, nor more than 6 ft. by 22 ft.

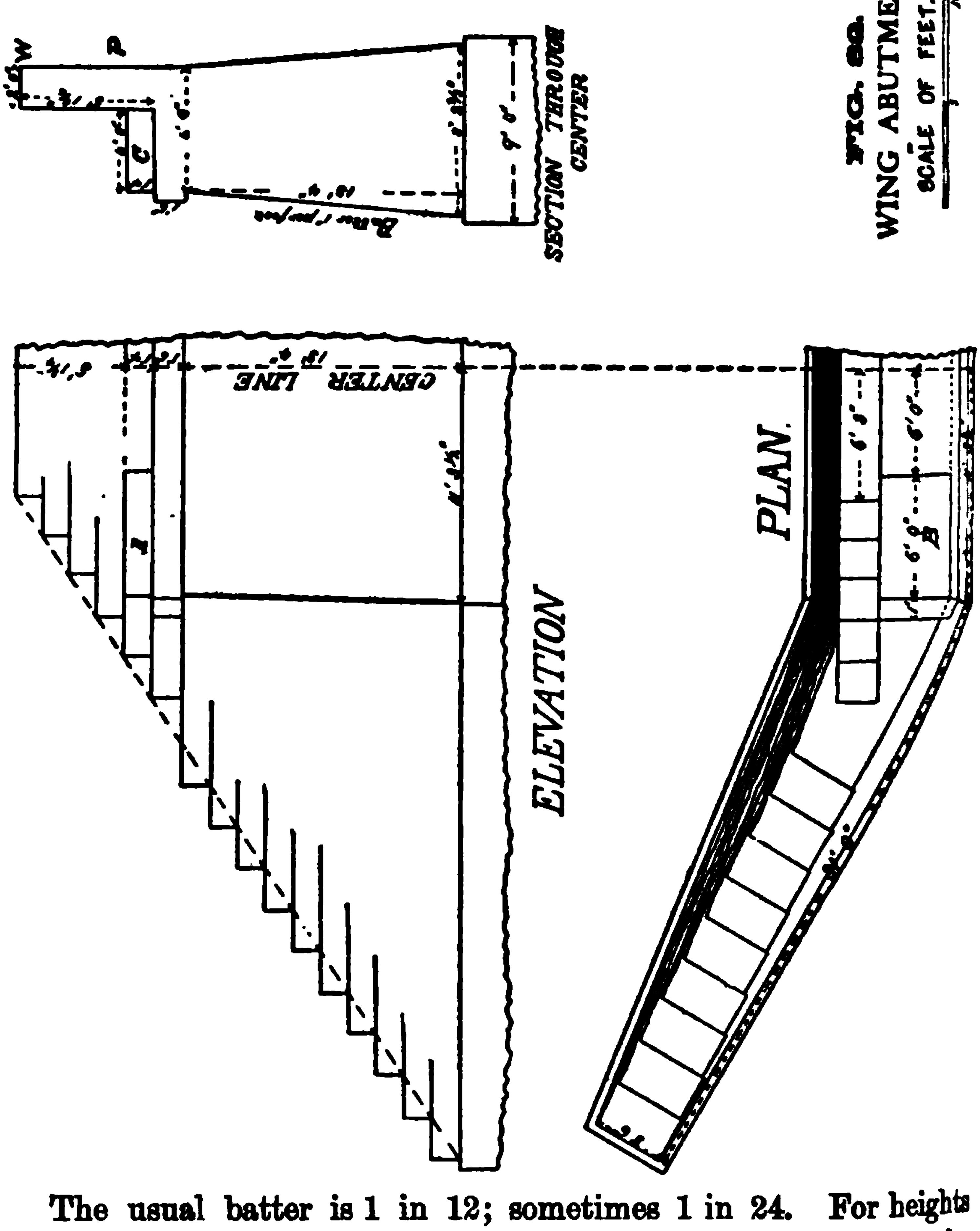

FIG. [illegible]. WING ABUTMENT.
SCALE OF FEET.

The usual batter is 1 in 12; sometimes 1 in 24. For heights under about 20 ft., the top dimensions and the batter determine the thickness at the bottom. For greater heights, the quite uniform

TABLE 37.

QUANTITY OF MASONRY IN WING ABUTMENTS OF THE GENERAL FORM SHOWN IN FIG. 82. SEE § 557.

Height of Abutment—Foundation to Coping.	Dimensions of the Bottom of Abutment.			Area of Lowest Course.			Masonry in one Abutment, exclusive of Footing, Copings, and Pedestals.*			
	Width of the Head.	Thickness of the Head.	Length of Face of Wing.	One Head.	Two Wings.	Total.	One Head Wall.	Two Wing Walls.	Parapet.	Total.
feet.	*feet.*	*feet.*	*feet.*	*sq ft.*	*sq. ft.*	*sq. ft.*	*cu. ft.*	*cu. ft.*	*cu. ft.*	*cu. yds.*
5	22.2	6.8	18.9	151	165	316	709	640	230	57.7
6	22.2	7.0	20.6	155	184	339	863	821	230	70.8
7	22.3	7.2	22.3	161	203	364	1,021	1,020	230	84.1
8	22.3	7.3	24.0	163	223	386	1,183	1,238	230	98.1
9	22.4	7.5	25.7	168	243	411	1,348	1,475	230	113.0
10	22.4	7.7	27.4	172	264	436	1,518	1,732	230	128.8
11	22.5	7.8	29.1	176	285	461	1,692	2,011	230	145.6
12	22.5	8.0	30.8	180	306	486	1,869	2,310	230	162.6
13	22.5	8.2	32.5	185	328	513	2 052	2,632	230	182.0
14	22.6	8.3	34.2	188	351	539	2,238	2,975	230	201.6
15	22.6	8.5	35.9	192	374	566	2,429	3,341	230	222.2
16	22.7	8.7	37.6	197	398	595	2,623	3,731	230	243.8
17	22.7	8.8	39.4	200	422	622	2,822	4,144	230	266.5
18	22.7	9.0	41.0	204	447	651	3,024	4,580	230	290.1
19	22.8	9.1	42.8	207	472	679	3,232	5,041	230	314.9
20	22.8	9.3	44.5	212	497	709	3,442	5,526	230	340.6
21	22.9	9.5	46.2	217	523	740	3,657	6,038	230	367.5
22	22.9	9.7	47.9	222	550	772	3,876	6,577	230	395.6
23	23.0	9.8	49.6	225	577	802	4,100	7,143	230	424.9
24	23.0	10.0	51.3	230	604	834	4,327	7,735	230	455.3
25	23.0	10.2	53.0	235	633	868	4,559	8,354	230	486.7
26	23.1	10.3	54.7	238	661	899	4,796	9,002	230	519.5
27	23.1	10.5	56.4	243	690	933	5,036	9,678	230	553.4
28	23.2	10.7	58.1	248	720	968	5,281	10,384	230	588.7
29	23.2	10.8	59.8	251	750	1,001	5,530	11,120	230	625.1
30	23.3	11.0	61.5	256	780	1,036	5,784	11,886	230	662.9
31	23.3	11.2	63.2	261	811	1,072	6,041	12,682	230	701.9
32	23.3	11.3	64.9	263	843	1,106	6,303	13,509	230	742.9
33	23.4	11.5	66.6	269	875	1,144	6,569	14,369	230	784.0
34	23.4	11.7	68.4	273	907	1,180	6,841	15,259	230	827.0
35	23.5	11.8	70.1	277	940	1,217	7,116	16,182	230	871.4
36	23.5	12.0	71.8	282	973	1,255	7,395	17,139	230	917.2
37	23.6	12.2	73.5	288	1,007	1,295	7,679	18,127	230	964.2
38	23.6	12.3	75.2	290	1,042	1,332	7,967	19,150	230	1,012.8

* Dimension stone in two pedestal blocks	=	64 cu. feet.
" " " coping of one abutment	=	234 " "
Total dimension stone in " "	=	298 " "

rule is to make the thickness four tenths of the height. The amount of masonry in the abutment is computed in accordance with this rule, although the actual quantity is usually more than that required by it. Since there is no objection to the wall's being rough, no

stones are cut out to secure the specified thickness, and hence the actual quantity of masonry usually exceeds the amount required. The spread of the footing courses and foundation will depend, of course, upon the location.

The wings should be proportioned according to the rules for retaining walls (see §§ 547 and 548). The wings are not always prolonged until their outer ends intersect the foot of the embankment slope; but are frequently stopped with an end height of 3 to 5 feet above the footing. The thickness of the wing wall decreases from the body of the abutment toward the tail in proportion to the height. For appearance, the top of the wing is usually made uniform from head to tail, being usually from 2½ to 3½ feet, according to the size of the structure. The steps should be capped with stones, not less than 1 foot thick, covering the entire step and extending under the step above not less than 1 foot.

557. Contents of Wing Abutments. The table on page 357 gives the quantities of masonry in wing abutments of the form shown in Fig. 82. Since the outlines of such structures are not simple geometrical figures, it is necessary to make more or less approximations in computing the cubical contents. For example, in Fig. 82 the wings are stepped off to fit the slope of the embankment as shown; and hence the corner of each course projects beyond the earthwork. The amount of masonry in these projecting corners varies as the thickness of the courses, and for any particular abutment it could be found accurately; but, in computing a table of general results, it is necessary to assume some thickness for the courses. In this case the courses were assumed to be 1 foot thick. The back of the "head" was assumed to conform strictly to the batter line, although in construction it would be stepped. The dimensions of the parapet wall will vary with the thickness of the pedestal blocks used, and also with the style of the bridge. The contents of the parapet as given in the table are for the dimensions shown in Fig. 82.

Footing courses were not included in the table, since they vary with the nature of the foundation. The area of the lowest course of masonry is given, from which the areas of the footing courses and of the foundation pit may be determined. The thickness at the top and the batter, as in Fig. 82, give, for any height found in the table, a thickness of wall at the bottom of at least four tenths of its

height (see § 548); for heights greater than in the table, the back of the wall must be stepped to keep the thickness four tenths of the height.*

558. U **Abutment.** Fig. 83 shows the standard plans of the Atchison, Topeka and Santa Fé R. R.† for U abutments. This is the only form of bridge abutment used on this road, except in special cases. The T abutment was once the standard, but was abandoned about fifteen years ago.‡

The specifications under which these abutments are built, require as follows: "1. Bed-plate pedestal blocks to be 2 feet thick, and placed symmetrically with regard to the plates. 2. Coping under pedestal blocks to be 18 inches thick for all spans exceeding 100 feet, 16 inches for 90 feet, and 14 inches for spans under 90 feet,—said coping to be through stones, and spaced alike from both sides of abutment. 3. Distances from front of dirt wall to front of bridge seat, and from grade line to top of bridge seat, and thickness of dirt wall, to vary for different styles and lengths of bridges. 4. Front walls to be 22 feet wide under bridge seat for all spans of 100 to 160 feet inclusive. 5. Total width of bridge seat to be $5\frac{1}{2}$ feet, for all spans. 6. Steps on back of walls to be used only when necessary to keep thickness $\frac{4}{10}$ of the height. 7. In case piling is not used, footing courses may be added to give secure foundation. 8. Length of wing walls to be determined by a slope of $1\frac{1}{2}$ to 1 at the back end of the walls—as shown by dotted line in front elevation,—thence by a slope of 1 to 1 down the outside —as shown on side elevation—to the intersection of the ground line with face of abutment. This rule may be modified in special cases. 9. Dimensions not given on the drawing are determined by the style and length of bridge, and are to be found on special sheet."

559. Although this road is noted for the excellency of its masonry, this design could be improved by leaving a weep hole in the side walls, 2 or 3 inches wide and the depth of a course of

* In computing the contents of masonry structures, it is necessary to remember that the volume of any mass which is made up of prisms, wedges, and pyramids—or cones—must be determined by the prismoidal formula; but if the mass is composed wholly of prisms and wedges, the contents can be correctly found by using the average of the end areas.

† Published by permission of A. A. Robinson, Chief Engineer.

‡ Compare with § 555.

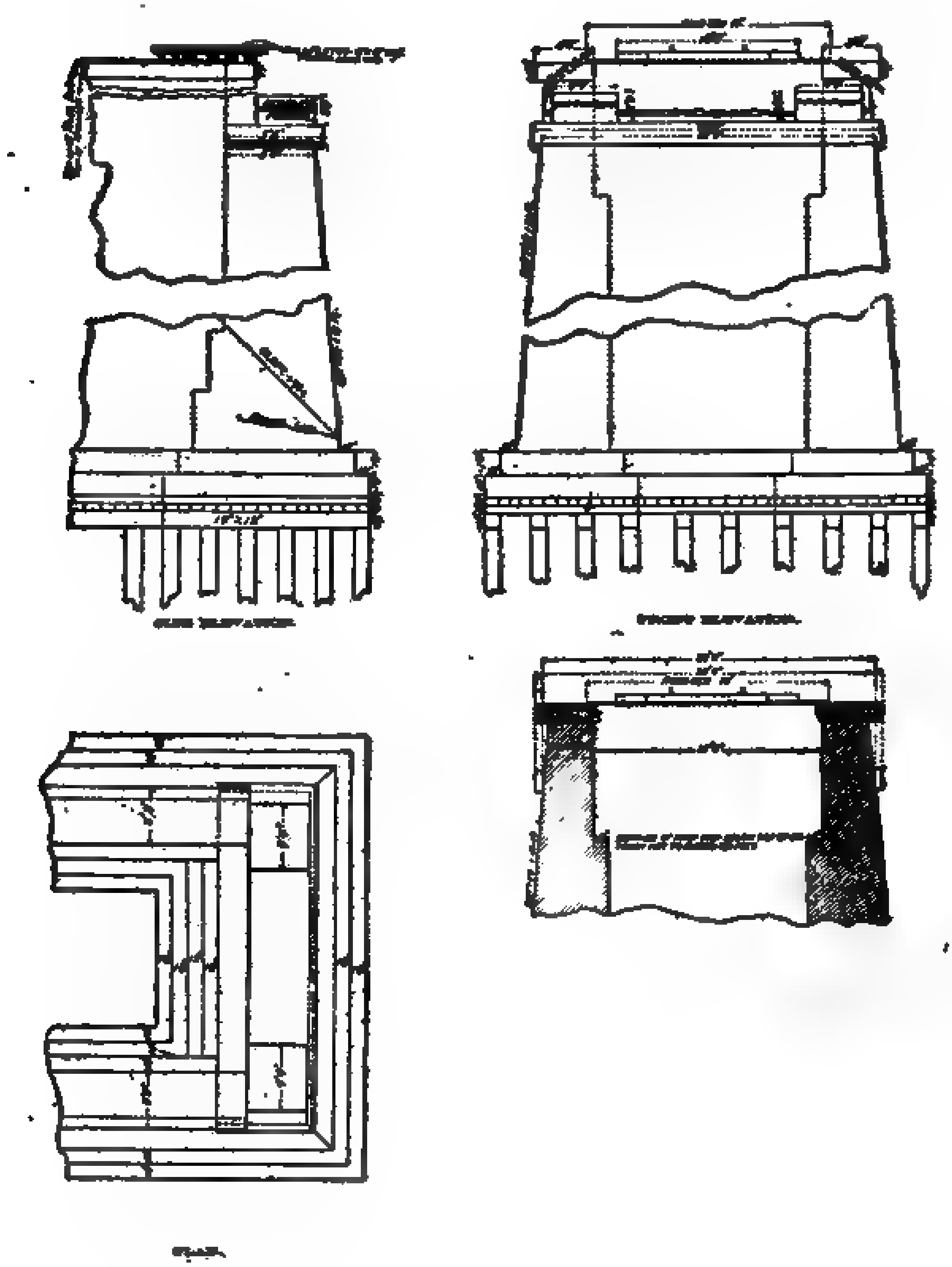

FIG. 68.—U ABUTMENT.—A. T. & S. F. R. R.

TABLE 33.

QUANTITY OF MASONRY IN U ABUTMENTS OF THE GENERAL FORM SHOWN IN FIG. 33. See § 560.

Height—Top of Footing to Bottom of Coping.	Dimensions of the Bottom of the Abutment.			Thickness of the Wing at the Bottom.	Quantity of Masonry, exclusive of Coping.*	
	Width.	Thickness.	Area.		Head.	Two Wings, per foot of length.
feet.	*feet.*	*feet.*	*feet.*	*feet.*	*cu. ft.*	*cu. ft.*
1	22.2	5.1	113	3.1	111	6.0
2	22.3	5.2	115	3.2	225	12.8
3	22.5	5.2	119	3.2	342	18.8
4	22.7	5.3	120	3.3	462	25.2
5	22.8	5.4	124	3.4	584	32.0
6	23.0	5.5	126	3.5	709	39.0
7	23.2	5.6	129	3.6	837	46.0
8	23.3	5.7	132	3.7	968	53.3
9	23.5	5.8	135	3.8	1,101	60.8
10	23.7	5.8	138	4.0	1,238	68.4
11	23.8	5.9	141	4.4	1,377	76.8
12	24.0	6.0	144	4.8	1,520	86.0
13	24.2	6.1	147	5.2	1,665	96.0
14	24.3	6.2	150	5.6	1,814	106.8
15	24.5	6.2	153	6.0	1,966	118.4
16	24.7	6.3	156	6.4	2,120	130.8
17	24.8	6.8	169	6.8	2,288	144.0
18	25.0	7.2	180	7.2	2,478	158.0
19	25.2	7.6	191	7.6	2,688	172.8
20	25.3	8.0	203	8.0	2,920	188.4
21	25.5	8.4	214	8.4	3,174	204.8
22	25.7	8.8	226	8.8	3,449	222.0
23	25.8	9.2	238	9.2	3,746	240.0
24	26.0	9.6	250	9.6	4,066	258.8
25	26.2	10.0	262	10.0	4,408	278.4
26	26.3	10.4	274	10.4	4,772	298.8
27	26.5	10.8	286	10.8	5,160	320.0
28	26.7	11.2	299	11.2	5,570	342.0
29	26.8	11.6	311	11.6	6,003	364.8
30	27.0	12.0	324	12.0	6,460	388.4
31	27.2	12.4	337	12.4	6,941	412.8
32	27.3	12.8	350	12.8	7,445	438.0
33	27.5	13.2	363	13.2	7,973	464.0
34	27.7	13.6	376	13.6	8,526	490.8
35	27.8	14.0	390	14.0	9,103	518.4

* For dimensions of coping and pedestal blocks, see second paragraph of § 558.

EXAMPLES OF THE METHOD OF USING THE TABLE.

Required the contents of the abutment in Fig. 33, assuming that the walls are continuous and not broken as shown.

Contents of dirt wall = 15 × 4 × 2	=	120.0 cu. ft.
" " 2 pedestal blocks = 2 × (2 × 4.33 × 4)	=	69.3 " "
" " coping on bridge seat = 23.0 × 5.5 × 1.50	=	189.7 " "
" " head (height = 20.25 ft.)	=	2,983.5 " "
" " 1st footing course = (24.7 + 2.0) × (6.4 + 2.0) × 1.5	=	336.4 " "
" " 2d " " = (24.7 + 4.0) × (6.4 + 4.0) × 1.5	=	447.7 " "
" " wing walls:—		
Above the bridge-seat coping (height = 3 ft.) = 18.8 × 2	=	37.6 " "
" " 1st step (" = 17 ") = 144 × 1	=	144.0 " "
" " 2d " (" = 21 ") = 204.8 × 1	=	204.8 " "
remaining 14 ft. of wings (height = 24.75 ft.) = 273.5 × 14	=	3,829.0 " "
" " coping on wings = 2 × (3.75 × 1.16 × 18.0)	=	155.1 " "
" " 1st footing under wings = (9.8 + 2.0) × 14 × 1.5	=	247.8 " "
" " 2d " " " = (9.8 + 4.0) × 14 × 1.5	=	289.8 " "
Total masonry = (335.4) cu. yds.	=	9,054.7 " "

masonry, for each 4 or 5 square yards of wing wall. Cinders, or sand and gravel are sometimes used to fill in between the wing walls to give a better drainage, and also to decrease the lateral thrust of the earth.

560. Contents of U Abutments. The table on page 361 gives the contents of U abutments of the form shown in Fig. 83. The

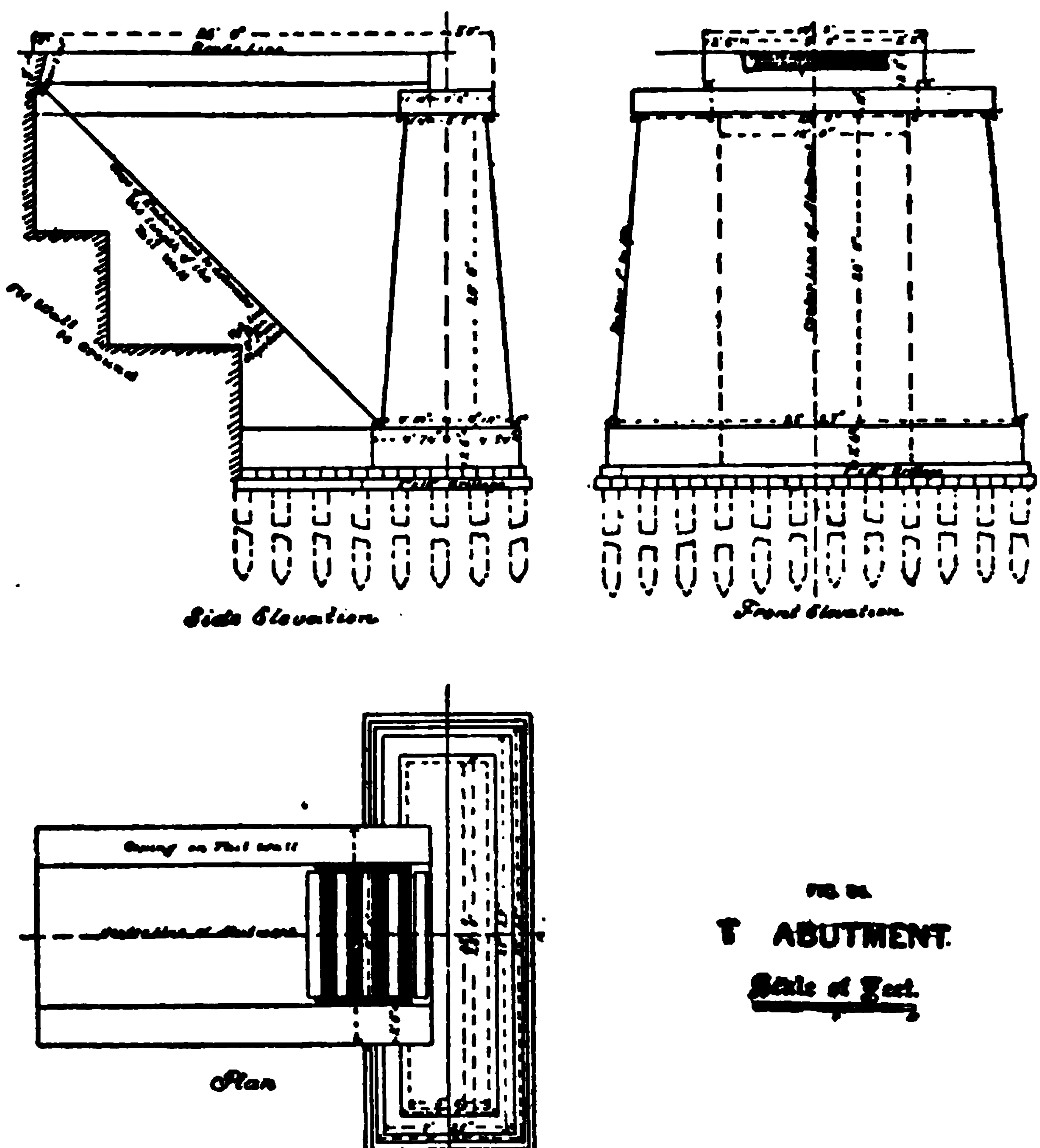

FIG. 84.
T ABUTMENT.
Scale of Feet.

quantities were computed on the basis that the thickness of the walls was four tenths the height, except that no wall was taken of a less thickness than that given by the thickness at the top and the batter as in the drawing.

561. T ABUTMENT. Fig. 84 shows the ordinary form of T abut-

TABLE 39.

QUANTITY OF MASONRY IN T ABUTMENTS OF THE GENERAL FORM SHOWN IN FIG. 84. SEE § 562.

Height of Abutment—Foundation to Coping.	Dimensions of the Bottom of the Head.			Quantity of Masonry, Exclusive of Coping.		
	Width.	Thickness.	Area.	Head.	Wedge.	Tail, per foot of length.
feet.	*feet.*	*feet.*	*feet.*	*cu. ft.*	*cu. ft.*	*cu. ft.*
5	[illegible]	5.8	133	607	12.5	60
6	23.0	6.0	138	[illegible]	18.0	72
7	23.2	6.2	[illegible]	883	24.5	84
8	23 3	6.3	[illegible]	1,029	32.0	[illegible]
9	23.5	6.5	153	1,179	40.5	108
10	23.7	6.7	158	1,334	50.0	[illegible]
11	23.8	6.8	163	1,495	60.5	[illegible]
12	24.0	7.0	[illegible]	1,660	72.0	144
13	24.2	7.2	173	1,831	84.5	156
14	24.3	7.3	178	2,006	98.0	[illegible]
15	24.5	7.5	[illegible]	2,188	112.5	[illegible]
16	24.7	7.7	189	2,374	128.0	192
17	24.8	7.8	[illegible]	[illegible]	144.5	204
18	25.0	8.0	[illegible]	2,763	162.0	216
19	25.2	8.2	206	2,966	180.5	228
20	[illegible]	8.3	211	3,174	200.0	240
21	25.5	8.5	217	3,388	[illegible]	252
22	25.7	8.7	[illegible]	3,606	242.0	264
23	25.8	8.8	228	[illegible]	264.5	276
24	26.0	9.0	234	4,064	288.0	288
25	26.2	9.2	240	4,301	312.5	300
26	[illegible]	9.3	246	4,544	[illegible]	312
27	[illegible]	9.5	252	4,793	[illegible]	[illegible]
28	26.7	9.7	258	5,047	[illegible]	336
29	[illegible]	9.8	[illegible]	5,306	420 5	[illegible]
30	27.0	10.0	270	5,575	450.0	360
31	27.2	10.2	[illegible]	5,848	480.5	372
32	27.3	10.3	282	6,127	512.0	384
33	27.5	10.5	289	6,413	[illegible]	396
34	27.7	10.7	295	6,705	578.0	[illegible]
35	27.8	10.8	301	7,003	612.5	[illegible]

Area of coping on 2 wings, per ft. of length = 5 sq. ft.
Area of coping on bridge-seat = 138 " "

EXAMPLE OF THE METHOD OF USING THE TABLE.

Required the contents of the abut … *sume the coping on the tail wall to be 1 foot thick, and also t* …

	cu. ft.
Contents of coping on wings = 5 sq. f … table) × 1 0 × 25.5 =	128
" " tail wall above lower … × 0.75 × 24.83 ft. =	261
" " " " " " … × 0 50 × 24.83 ft. =	169
" " " " " " " " " = 13.0 × 1.50 × 23.00 ft. =	448
" " wall to keep back ballast = 9 0 × 1.0 × 1.5 =	14
" " coping on bridge-seat = 138 (see foot of table) × 1.5 =	207
" " head (from the table for height = 20 ft.) =	3,174
" " wedge " " " " " = 20 ft.) =	200
" " lowest step of tail, (height = 20 ft., length = 8.84 ft.) = 240 × 8 84 =	2,122
" " middle " " " " = 15 ft., " = 8.5 ft.) = 180 × 8 5 =	1,530
" " top " " " " = 8 ft., " = 4 5 ft.) = 96 × 4.5 =	432
" " footing under head = 9.4 × 26 2 × 2.5 ft =	616
" " " " tail = 8.4 × 13.0 × 2.5 ft =	273

Total masonry = 354.3 cubic yards = 9,567

ment. For railroad bridges the head is usually not less than 5 ft. × 20 ft., nor more than 6 ft. × 22 ft., under the coping, according to the size of the bridge. The tail wall is usually 10 or 12 ft. wide, and of such length that the foot of the slope of the embankment will just reach to the back of the head wall. The batter on the head wall is 1 to 12 or 1 to 24 all around. The tail wall is generally built vertical on the sides and the end. Notice the batter at the top of the free end of the tail wall. This is known as the "frost batter," and is to prevent the frost from dislocating the corner of the masonry. The drainage of the ballast pocket should be provided for by leaving a space between the ends of two stones. Formerly the tail wall was sometimes only 7 or 8 feet wide, in which case the ties were laid directly upon the masonry without the intervention of ballast; but this practice has been abandoned, as being very destructive of both rolling stock and masonry.

According to the common theories for retaining walls, T abutments with dimensions as above have very large factors of stability against sliding, and overturning, and crushing.

562. Contents of T Abutments. The table on page 363 gives the contents of the abutments of the form shown in Fig. 84. The height of the tail above the under side of the bridge-seat coping will vary with the thickness of the pedestal blocks, and with the style of the bridge; and hence the table gives the quantities in the abutment below the bridge-seat coping and above the footing. The quantity of masonry above this line will vary also with the amount of ballast used. The term "wedge" in the table is used to designate that part of the tail included between the head and a vertical plane through the lower edge of the back face of the head.

563. Foundation. Usually but little difficulty is encountered in securing a foundation for bridge abutments. Frequently the foundation is shallow, and can be put down without a coffer-dam, or at most with only a light curb (see §§ 316–20). Where the ground is soft or liable to scour, a pile foundation and grillage is generally employed. For the method of doing this, see Art. 3, Chapter XI; and for examples of this kind of foundation, see Fig. 84 (page 362), Fig. 86 (page 380), and Fig. 90 (page 386).

Where there is no danger of underwashing, and where the foundation will at all times be under water, the masonry may be started upon a timber platform consisting of timbers from, say, 8 to 12

inches thick, laid side by side upon sills, and covered by one or more layers of timbers or thick planks, according to the depth of the foundation and the magnitude of the structure. For an example of a foundation of this class, see Plate II. For a discussion of the method of failure by sliding on the foundation, see § 491.

564. Quality of Masonry.—Bridge abutments are built of first-class masonry (§ 207) or of second-class (§§ 209 and 212), according to the importance of the structure. See also the specifications for bridge pier masonry (§§ 591–600). The coping should be composed of as large stones as practicable—not less than 12 inches thick, and 15 or 18 inches thick is better and more frequently used.

Sometimes, the bed plates of the bridge rest directly upon the coping, but usually upon a stone pedestal block (see Figs. 82 and 83), in which case small pedestals, upon which the rail stringers rest (see Fig. 90, page 386), are also generally used.

565. Cost. For data on the cost of masonry, see §§ 232–38.

CHAPTER XVI.

BRIDGE PIERS.

566. The selection of the site of the bridge and the arrangement of the spans, although important in themselves, do not properly belong to the part of the problem here considered; therefore they will be discussed only briefly. The location of the bridge is usually a compromise between the interests of the railroad or highway, and of the river. On navigable streams, the location of a bridge, its height, position of piers, etc., are subject to the approval of engineers appointed for the purpose by the United States Government. The law requires that the bridge shall cross the main channel nearly at right angles, and that the abutments shall not contract nor the piers obstruct the water way. For the regulations governing the various streams, and also reports made on special cases, see the various annual reports of the Chief of Engineers, U. S. A., particularly Appendix X_2, of the Report for 1878.

The arrangement of the spans is determined mainly by the relative expense for foundations, and the increased expense per linear foot of long spans. Where the piers are low and foundations easily secured, with a correspondingly light cost, short spans and an increased number of piers are generally economical, provided the piers do not dangerously obstruct the current or the stream is not navigable. On the other hand, where the cost of securing proper foundations is great and much difficulty is likely to be encountered, long spans and the minimum number of piers is best. Sound judgment and large experience are required in comparing and deciding upon the plan best adapted to the varying local conditions.

Within a few years it has become necessary to build bridge piers of very great height, and for economical considerations iron has been substituted for stone. The determination of the stability of such piers is wholly a question of finding the stress in frame structures,—the consideration of which is foreign to our subject.

ART. 1. THEORY OF STABILITY.

567. METHOD OF FAILURE. A bridge pier may fail in any one of three ways: (1) by sliding on any section on account of the action of the wind against the train, bridge, and exposed part of the pier, and of the current of the stream against the immersed part of the pier; or (2) by overturning at any section when the moment of the horizontal forces above the section exceeds the moment of the weight on the section; or (3) by crushing at any section under the combined weight of the pier, the bridge, and the train. The dimensions of piers are seldom determined by the preceding conditions; the dimensions required at the top (§ 584) for the bridge seat, together with a slight batter for appearance, generally give sufficient stability against sliding, overturning, and crushing. However, the method of determining the stability will be briefly outlined and illustrated by an example.

568. STABILITY AGAINST SLIDING. Effect of the Wind. The pressure of the wind against the truss alone is usually taken at 50 lbs. per sq. ft. against twice the vertical projection of one truss, which for well-proportioned iron trusses will average about 10 sq. ft. per linear foot of span. The pressure of the wind against the truss and train together is usually taken at 30 lbs. per sq. ft. of truss and train. The train exposes about 10 sq. ft. of surface per linear foot. The pressure of the wind against any other than a flat surface is not known with any certainty; for a cylinder, it is usually assumed that the pressure is two thirds of that against its vertical projection.

569. Effect of Current. For the pressure of the current of water against an obstruction, Weisbach's Mechanics of Engineering (page 1,030 of Coxe's edition) gives the formula,

$$P = s\,w\,k\,\frac{v^2}{2g}, \quad \ldots \ldots \ldots \quad (1)$$

in which P is the pressure in pounds, s the exposed surface in sq. ft., k a co-efficient depending upon the ratio of width to length of the pier, w the weight of a cubic foot of water, v the velocity in ft. per sec., and g the acceleration of gravity. For piers with rectangular cross section, k varies between 1.47 and 1.33, the first being for square piers and the latter for those 3 times as long as

wide; for cylinders, k = about 0.73. The law of the variation of the velocity with depth is not certainly known; but it is probable that the velocity varies as the ordinates of an ellipse, the greatest velocity being a little below the surface. Of course, the water has its maximum effect when at its highest stage.

570. Effect of Ice. The pier is also liable to a horizontal pressure due to floating ice. The formulas for impact are not applicable to this case. The assumption is sometimes made that the field of ice which may rest against the pier, will simply increase the surface exposed to the pressure of the current. The greatest pressure possible will occur when a field of ice, so large that it is not stopped by the impact, strikes the pier and plows past, crushing a channel through it equal to the greatest width of the pier. The resulting horizontal pressure is equal to the area crushed multiplied by the crushing strength of the ice. The latter varies with the temperature; but since ice will move down stream in fields only when melting, we desire its minimum strength. The crushing strength of floating ice is sometimes put at 20 tons per sq. ft. (300 lbs. per sq. inch); but in computing the stability of the piers of the St. Louis steel-arch bridge, it was taken at 600 lbs. per sq. inch (43 tons per sq. ft.). According to some German experiments, the crushing strength of ice, at 32° F., varies between 61 and 204 lbs. per sq. in.*

Occasionally a gorge of ice may form between the piers, and dam the water back. The resulting horizontal pressure on a pier will then be equal to the hydrostatic pressure on the width of the pier and half the span on either side, due to the difference between the level of the water immediately above and below the bridge opening. A pier is also liable to blows from rafts, boats, etc.; but as these can not occur simultaneously with a field of ice, and will probably be smaller than that, it will not generally be necessary to consider them.

A lateral pressure on the pier is possible, due to the earth's being washed away from one side and not from the opposite. It will be on the safe side, and near enough for this purpose, to assume that this effect is equal to the pressure of a liquid whose density is the difference between that of the water and the saturated soil displaced. Under these conditions, the actual tendency to slide is

* *Engineering News*, vol. xiv. p. 407.

equal to the square root of the sum of the down stream forces and the lateral thrust. However, this refinement is unnecessary, particularly since a pier which is reasonably safe against overturning and crushing will be amply safe against sliding.

571. Resisting Forces. The resisting force is the friction due to the combined weight of the train, bridge, and the part of the pier above the section considered. For the greatest refinement, it would be necessary to compute the forces tending to slide the pier for two conditions: viz., (1) with a wind of 50 lbs. per sq. ft. on truss and pier, in which case the weight of the train should be omitted from the resisting forces; and (2) with a wind of 30 lbs. per sq. ft. on truss, train, and pier, in which case the weight of a train of *empty box* cars should be included in the resisting forces. For a table of weights of masonry, see page 200. If the water can find its way under the foundation in thin sheets, the weight of the part of the pier that is immersed in the water will be diminished by 62½ lbs. per cu. ft. by buoyancy; but if it finds its way under any section by absorption only, then no allowance need be made for buoyancy.

The resisting force is equal to the product of the total weight and the co-efficient of friction. For values of the co-efficient of friction, see the table on page 315. The tenacity of the mortar is usually neglected, although it is a very considerable element of strength (see § 137).

572. STABILITY AGAINST OVERTURNING. The forces which tend to produce sliding also tend to produce overturning, and the forces which resist sliding also resist overturning; hence, there remains to determine only their points of application. The stability can be determined either by moments or by resolution, as was explained for dams; but in this case, it is easier by moments, since there are several horizontal forces, and it requires considerable work to find their resultant as demanded by the method by resolution of forces.

573. A. By Moments. By this method, it is necessary to find the arm of the forces, *i. e.*, the perpendicular distance from the line of action of the forces to a point about which the pier tends to turn. This is the same method as that used in §§ 493–98, which see.

The center of pressure of the wind on the truss is practically at the middle of its height; that of the wind on the train is 7 to 9 feet above the top of the rail; and that of the wind on the pier is at the middle of the exposed part. The arm for the pressure of the

ice should be measured from high water? The center of pressure of the current is not easily determined, since the law of the variation of the velocity with the depth is not known; but it will probably be safe to take it at one third the depth. Finally, the downward forces will usually act vertically through the center of the pier.

From these data the overturning and resisting moments can easily be computed. For equilibrium, the summation of the former must be less than the latter. The above principles will be further elucidated in §§ 579–80 by an example.

574. B. By Resolution of Forces. This is the method explained in § 499 (page 320). In that case the problem was very simple, since there were but two forces; but in the present case there are several horizontal forces and also several vertical ones. The first step is to find a single force which is equivalent in every respect to the combined effect of all the horizontal forces; the second is to find an equivalent for all of the vertical forces; and the third is to find the resultant of these two forces.

The horizontal distance, x, of the point of application of the resultant of all the vertical forces, back from the toe of the pier, is found by the equation,

$$x = \frac{\textit{sum of the moments of the vertical forces}}{\textit{sum of the vertical forces}}. \quad . \quad . \quad (2)$$

The weight of the train and bridge act vertically through the center of the pier; and if the pier is symmetrical, as it usually is, the weight of the pier will also act through its center. Therefore, x in equation (2) will usually be half the length of the pier.

The vertical distance, y, of the point of application of the resultant of all the horizontal forces above any horizontal joint is found by the equation,

$$y = \frac{\textit{sum of the moments of the horizontal forces}}{\textit{sum of the horizontal forces}}. \quad . \quad . \quad (3)$$

Having found x and y, as above, draw a vertical line at a distance x back from the down stream end of the pier; on this line lay off a distance y above the horizontal joint under consideration. The point thus determined corresponds to a of Fig. 70 (page 320). Construct the parallelogram of forces by laying off, to any convenient

scale, (1) a horizontal line equal to the sum of all the horizontal forces acting on the pier, and (2) a vertical line equal to the sum of all the vertical forces; and complete the diagram by drawing the resultant. The stability of the pier is determined by the ratio of AC to NC, Fig. 70.

575. Stability against Crushing. Represent the maximum pressure by P, the total weight on the section by W, the area of the section by S, the moment of inertia of the section by I, the length of the section by l, and the overturning moment by M; then from equation (1), page 205, we have

$$P = \frac{W}{S} + \frac{M l}{2 I} \quad \ldots \ldots \ldots \quad (4)$$

For the particular case in which the pier has a rectangular horizontal cross section, the above formula becomes the same as equation (18), (page 322,) as deduced for an element of a masonry dam.

The method of applying the above equation will be explained in § 581 by an example.

576. Example of Method of Computing Stability. Fig. 85 shows the dimensions of the channel pier of the Illinois Central R. R. bridge over the Ohio River at Cairo, Ill. This pier stands between two 523-foot spans. Its stability will now be tested by the preceding principles.

577. Stability against Sliding. We will examine the stability against sliding on the top footing course. The wind surface of the truss = 10 sq. ft. × 523 = 5,230 sq. ft. The wind pressure against the truss at 30 lbs. per sq. ft. = 30 lbs. × 5,230 =156,900 lbs. = 78 tons; and the wind pressure on the truss at 50 lbs. = 50 lbs. × 5,230 = 261,500 lbs. = 131 tons.

The wind pressure on train at 30 lbs. per sq. ft. = 30 lbs. × 523 × 10 = 156,900 lbs = 78 tons.

The pressure of the wind against a section of the pier 52 ft. long, is 20 lbs. × 52 × 14 = 14,560 lbs. = 7 tons.

The pressure due to the ice is found as follows: Assume the thickness to be 1 foot, and also assume the crushing strength of ice to be 200 lbs. per sq. in. =, say, 15 tons per sq. ft. The pier is 16 ft. wide at the high-water line. Hence the resistance required in the pier to crush its way through a field of ice is 15 tons × 16 × 1 = 240 tons.

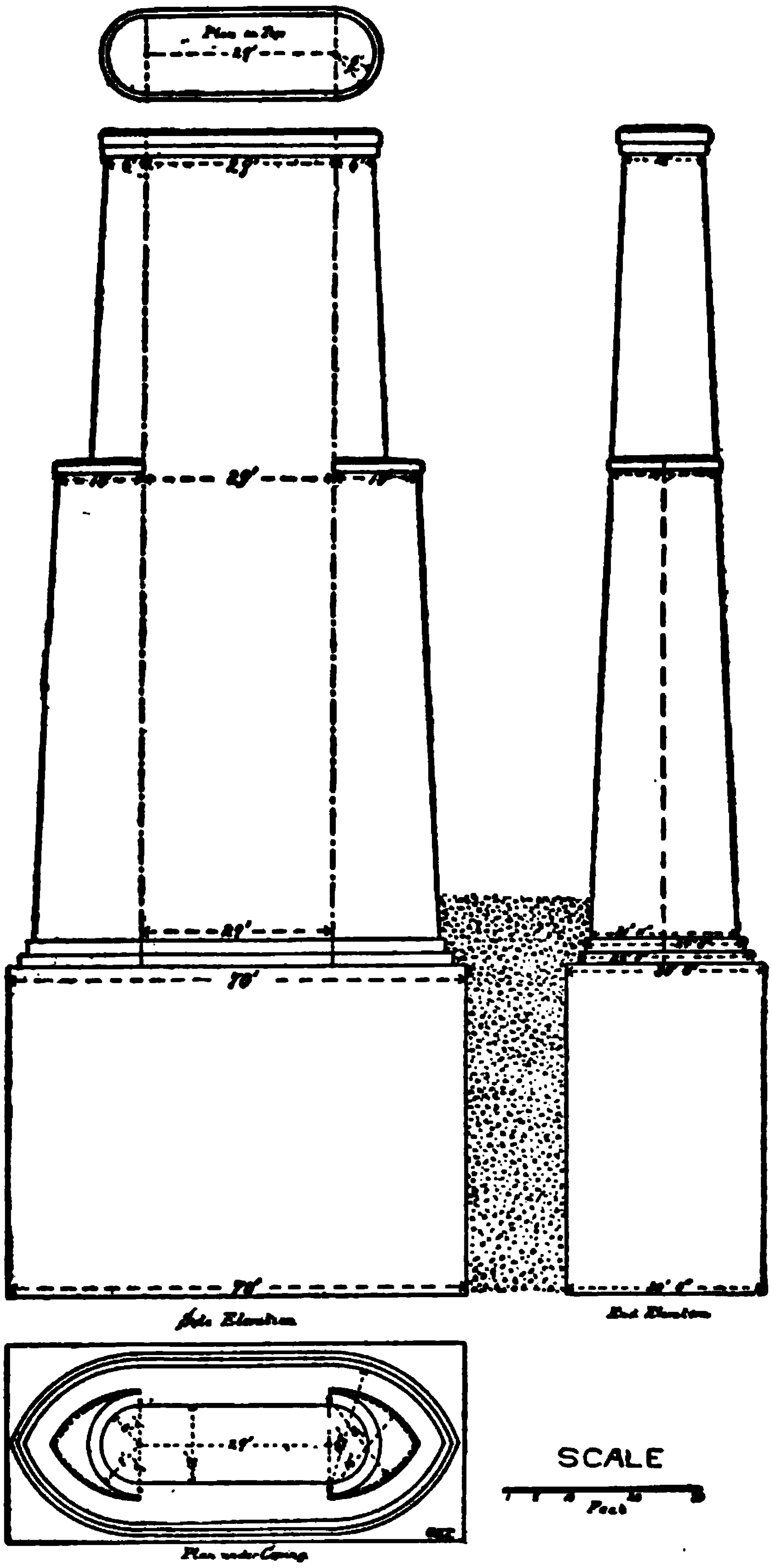

FIG. 85.—CHANNEL PIER, CAIRO BRIDGE.

The pressure due to the current is found as follows: From § 569, $P = s\,w\,k\,\frac{v^2}{2g}$. s represents the exposed surface = 70 ft. × 19 ft. = 1,330 sq. ft., which value is equivalent to assuming that the river may scour to the top of the footing courses. k represents a co-efficient, which, if the pier were rectangular, would be about 1.4, and if the pier were cylindrical would equal about 0.73. We will assume it to be 1.1,—a trifle more than the mean of these two values. $w = 62.5$ lbs. per cu. ft. The surface velocity at the bridge site was measured* "when the Mississippi and the Ohio were at about the same stage," and found to be 4 miles per hour (= 6 ft. per second); but as high water may occur in the Ohio at the time of moderately low water in the Mississippi, the possible maximum velocity is greater than the above, and hence we will assume that it is 10 ft. per second. The velocity of the water at various depths below the surface of a stream varies as the ordinate of an ellipse; but the effect of the mean velocity is approximated with sufficient accuracy for this purpose by assuming that the mean pressure is half of that due to the surface velocity. Substituting these numbers, the above equation becomes $P = 1{,}330 \times 1.1 \times 62.5 \times \frac{100}{64} = 70.5$ tons = 70 tons with sufficient accuracy. Dividing this by 2 to get the pressure corresponding to the mean velocity, we have the pressure of the current equal to 35 tons.

Collecting the preceding results, we have:

Wind on truss, ...	78 tons.
" " train, ...	78 "
" " pier, ...	7 "
Pressure of ice, ...	240 "
" " water, ...	35 "
Total force tending to slide the pier on the footing . . . . =	438 tons.

578. The weight of the bridge will be assumed at 2 tons per lineal foot; and hence the total weight is 2 tons × 523 = 1,046 tons.

The weight of a train of *empty* cars is about 0.5 ton per lineal

* Third Annual Report of the Illinois Society of Engineers, p. 78.

foot; and hence the total weight of the train is 0.5 tons × 523 = 261 tons.

The amount of masonry below the high-water line = 67,946 cu. ft.; the amount above the high water line = 24,534 cu. ft.; and hence the total masonry = 92,480 cu. ft. We will assume the weight of the masonry to be 150 lbs. per cubic foot. Then the weight of the masonry is 150 lbs. × 92,480 = 6,936 tons.

Collecting these results, we have:

Weight of the bridge,	. . . .	1,046 tons.
" " " train of empty cars,	. . . . .	261 "
" " " masonry,	. . . .	6,936 "
Total weight to resist sliding	. . . . . . =	8,243 tons.

Sliding cannot take place, if the co-efficient of friction is equal to or greater than 438 ÷ 8,243 = 0.053. For values of the co-efficients of friction, see the table on page 315. In the above example, the factor of safety against sliding is at least 12 to 15.

579. Stability against Overturning. We will consider the stability against overturning about the top of the upper footing course. The wind on the truss = 78 tons; the arm of this force = *height of the pier* (123 ft.) + *half the depth of the truss* (30 ft.) = 153 ft.; and therefore the moment of this force = 78 tons × 153 ft. = 11,934 foot-tons.

The pressure of the wind on the train = 78 tons; and the arm of this pressure = *distance from footing to top of pier* (123 ft.) + *distance from top of pier to top of rail* (8 ft.) + *distance from top of rail to center of train* (8 ft.) = 139 ft. Therefore the moment of this pressure is 78 tons × 139 ft. = 10,842 foot-tons.

The pressure of the wind against the pier is 7 tons (§ 577); the arm of this force = $\frac{1}{2}$ (202 + 150) − 79 = 97 ft.; and the moment of this force = 679 foot-tons.

The pressure of the ice is 240 tons, the arm is 70 ft., and the moment is 16,800 foot-tons.

The pressure of the water is 35 tons. The center of pressure lies somewhere between one third and one half of the depth from the top; and as the increased area at the base of the pier compensates in part for the decrease of velocity with the depth, we will assume that it is at half the depth. The arm then is 36 ft., and the moment is 35 tons × 36 ft. = 1,260 foot-tons.

Collecting these results, we have:

Moment of the wind on the truss, . . .	11,934 foot-tons.
" " " " " " train, . . .	10,842 "
" " " " " " pier, ...	679 "
" " " pressure of the ice, . . .	16,800 "
" " " " " " current . .	1,260 "
Total overturning moment . . . . . . =	41,515 foot-tons.

580. The total weight above the joint considered is (§ 578) 8,243 tons. This force acts vertically down through the center of the pier; hence the arm is 31.5 ft., and the total moment resisting overturning is $8,243 \times 31.5 = 259,654$ foot-tons. The factor of safety against overturning about the top of the upper footing course is $259,654 \div 41,515 = 6.3$.

Assuming the train to be off the bridge, and that the wind pressure on the truss is 50 lbs. per sq. ft., and following the method pursued above, it is found that the factor of safety against sliding under these conditions is 6.4.

581. Stability against Crushing. The maximum pressure on the section will occur when the loaded train is on the bridge and all the horizontal forces are acting with their full intensity. The load when an *empty* train is on the bridge is (§ 578) 8,243 tons. Assuming that a loaded train will weigh $1\frac{1}{4}$ tons per lineal foot, we must add (0.75 tons $\times 523 =$) 392 tons to the above for the difference between a loaded and an unloaded train. Then the total direct pressure is $8,243 + 392 = 8,635$ tons. The area of the section at the top of the footing course is 1,160 sq. ft. Hence, the maximum direct pressure is $8,635 \div 1,160 = 7.4$ tons per sq. ft.

The moment to overturn, M, $= 41,515$ foot-tons. The greatest length of the section $= 63$ ft. The moment of inertia of the section about an axis through its center and perpendicular to its length $= 287,917$ (ft.). From § 575, the maximum pressure

$$P = \frac{W}{S} + \frac{Ml}{2I}.$$

Substituting the above quantities in this equation gives

$$P = 7.4 + \frac{41,515 \times 63}{2 \times 287,917} = 7.4 + 4.5 = 11.9 \text{ tons per sq. ft.}$$

Since it is highly improbable that all the forces will act at the same time with the intensity assumed in the preceding computa-

tions, we may conclude that the pressure will never exceed 11.9 tons per sq. ft. A comparison of this with the values of the compressive strength of masonry as given in § 222 (page 149) shows that this pressure is entirely safe.

Since this is an unusually high pier under an unusually long span, and since the overturning and resisting moments and also the top dimensions of the pier vary with the span, we may draw the conclusion that *any pier which has sufficient room on top for the bridge seat* (§ 584) *and which has a batter of* 1 *in* 12, *or* 1 *in* 24, *is safe against any mode of failure.*

582. Pressure on the Bed of the Foundation. The caisson is 70 feet long, 30 feet wide, and 50 feet high. The load on the base is equal to the weight on the top of the footing *plus* the weight of the footings *plus* the weight of the caisson. The weight above the footing = 8,635 tons (§ 581). The weight of the footings = 1,300 sq. ft. × 4 ft. × 150 lbs. = 390 tons. The weight of the caisson = 70 ft. × 30 ft. × 50 ft. × 100 lbs. = 5,250 tons. The total weight on the bed = 8,635 + 390 + 5,250 = 14,275 tons. The area = 70 ft. × 30 ft. = 2,100 sq. ft. The *direct* pressure per unit of area = 14,275 ÷ 2,100 = 6.8 tons per sq. ft.

The overturning moment, M, is equal to the moment about the top of the footing (§ 581) *plus* the product of the sum of the horizontal forces and the distance from the footing to the base of the caisson; or, the moment about the base = 41,515 foot-tons + 438 tons × 54 ft. = 65,167 foot-tons. The moment of inertia, $I = \frac{1}{12}\, 30\,(70)^3 = 857{,}500$ (ft.). $l = 70$ ft. The concentrated pressure caused by the tendency to overturn is

$$\frac{M\,l}{2\,I} = \frac{65{,}167 \times 70}{2 \times 857{,}500} = 2.7 \text{ tons.}$$

The caisson was sunk all the way through, and rests, on sand; consequently the water will find its way freely under the entire foundation, thus causing buoyancy to act with its full force. This upward force of the water will be equal to the volume of the immersed masonry *multiplied by* the weight of a cubic foot of water; or the buoyancy = (67,946 + 5,200 + 105,000) × 62.4 = 5,558 tons. The lifting effect of buoyancy is (5,558 ÷ 2,100 =) 2.62 tons per sq. ft.

Therefore, the total pressure is not greater than 6.8 + 2.7 − 2.6 = 6.9 tons per sq. ft.

The pressure would never be so much, for the following reasons: 1. There is no probability that both spans will be covered by a train of maximum weight at the same time that the maximum effects of the wind, of the current, and of the ice occur. 2. The friction on the sides of the caisson will sustain part of the load. A friction of 600 lbs. per sq. ft., which was about the amount experienced in sinking these piers (see § 455), would decrease this pressure about 1¼ tons per sq. ft.

Therefore, we conclude that the pressure on the sand will be at least as much as $6.8 - 1.5 - 2.6 = 2.7$ tons per sq. ft.; and that it may possibly, but not probably, amount to $6.8 + 2.7 - 2.6 - 1.5 = 5.4$ tons per sq. ft. The larger value was taken at the greatest possible one for the sake of establishing the conclusion stated in the last paragraph of § 581.

583. *Other Examples.* At the St. Louis steel-arch bridge the greatest pressure possible on the deepest foundation (bed-rock) is 19 tons per sq. ft. The pressure at the base of the New York tower of the East River suspension bridge is about 7½ tons per sq. ft. upon a stratum of sand 2 feet thick overlying bed-rock; and at the base of the masonry the pressure is about 11¼ tons per sq. ft.* The corresponding quantities for the Brooklyn tower were a little over a ton less in each case. At the Plattsmouth bridge † the maximum pressure caused by the weight of train, bridge, and pier is 3 tons per sq. ft. At the Bismarck bridge † the pressure due to the direct weight is 3 tons per sq. ft. on clay.

ART. 2. DETAILS OF CONSTRUCTION.

584. TOP DIMENSIONS. The dimensions on the top will depend somewhat upon the form of the cross section of the pier, and also upon the style and span of the bridge; but, in a general way, it may be stated that, for trussed spans of 100 ft. or over, the dimensions under the coping will not be less than 5 ft. × 20 ft.; for 250-ft. spans, 8 ft. × 30 ft.; and for 500-ft. spans, 10 ft. × 40 ft. Apparently 6 ft. × 22 ft. under the coping is the favorite size for spans of 100 to 200 ft.

* F. Collingwood, assistant engineer, in Van Nostrand's Engin'g Mag., vol. xvi. p. 431.

† Report of Geo. S. Morison, chief engineer.

585. Bottom Dimensions. Theoretically the dimensions at the bottom are determined by the area necessary for stability; but the top dimensions required for the bridge seat, together with a slight batter for the sake of appearance, gives sufficient stability (§ 581). Only high piers for short spans—a combination not likely to occur in practice—are liable to fail by overturning or crushing.

586. Batter. The usual batter is 1 inch to a foot, although ½ an inch to a foot is very common. In high piers it is customary to use a batter of 1 to 24, and offset the masonry and introduce a water-table at the high-water line, so as to give an average batter of about 1 to 12. This construction very much improves the appearance, and does not add materially to the cost.

A corbel course, or "belt course," is sometimes introduced immediately under the coping for appearance's sake. For an example, see Fig. 85 (page 372), Fig. 87 (page 383), and Fig. 88 (page 384).

587. Cross Section. The up-stream end of a pier, and to a considerable extent the down-stream end also, should be rounded or pointed to serve as a cut-water to turn the current aside and to prevent the formation of whirls which act upon the bed of the stream around the foundation, and also to prevent shock from ice, logs, boats, etc. In some respects the semi-ellipse is the best form for the ends; but as it is more expensive to form, the ends are usually finished to intersecting arcs of circles (see Figs. 85, 87, and 89—pages 372, 383, and 385, respectively), or with semi-circular ends. Above the high-water line a rectangular cross section is as good as a curved outline, except possibly for appearance.

A cheaper, but not quite as efficient, construction is to form the two ends, called starlings, of two inclined planes. As seen in plan, the sides of the starlings usually make an angle of about 45° with the sides of the pier (see Fig. 90, page 386). A still cheaper construction, and the one most common for the smaller piers, is to finish the up-stream end, below the high-water line, with two inclined planes which intersect each other in a line having a batter of from 3 to 9 inches per foot, and build the other three sides and the part of the up-stream face above the high-water line with a batter of 1 in 12 or 1 in 24. Of course the simplest construction is to make the pier rectangular in horizontal cross sections and give it the same batter on all faces.

Occasionally, for economy, piers, particularly pivot piers, are built hollow—sometimes with and sometimes without interior cross walls (see Fig. 86, page 380). The piers of the bridge across the Missouri River at Glasgow, Mo., are solid up to the high-water line, and above that each pier consists of two stone columns. The piers of the bridge over the Missouri at St. Charles, Mo., have a somewhat similar construction, except that the secondary piers are connected by a comparatively thin wall.

With piers subjected to a severe pressure from ice, it is customary to protect the edge of the nose with an angle-iron or a railroad rail.

588. Pivot Piers. These differ from the ordinary piers only in that they are circular, are larger on top, and have plumb sides. Pivot piers are about 25 to 30 feet in diameter, under the coping, for spans of 250 to 350 feet, respectively.

Fig. 86 shows the pivot pier for the Northern Pacific R. R. bridge over the Red River at Grand Forks, Dakota. The specifications for the grillage were as follows: "Fasten the first course of timbers together with $\frac{3}{4}$-inch × 20-inch drift bolts, 18 inches apart; fasten second course to first course with drift bolts of same size at every other intersection. Timbers to be laid with broken joints. Put on top course of 4-inch × 12-inch plank, nailed every 2 feet with $\frac{7}{16}$-inch × 8-inch boat spikes. The last course is to be thoroughly calked with oakum."

Pivot piers are protected from the pressure of ice and from shock by boats, etc., by an ice breaker which is entirely distinct from the pier. The ice breaker is usually constructed by driving a group of 60 or 70 piles in the form of a V (the sharp end up stream), at a short distance above the pier. On and above these piles a strong timber crib-work is framed so as to form an inclined ridge up which the cakes of ice slide and break in two of their own weight. Between the ice breaker and the pier two rows of piles are driven, on which a comparatively light crib is constructed for the greater security of the pier and also for the protection of the river craft.

589. Quality of Masonry. Bridge piers are usually quarry-faced ashlar, *i. e.*, first-class masonry (see § 207) backed with rubble. Good concrete, if made with reasonable care, is equally as good as ordinary rubble masonry, and is sometimes cheaper,—since it affords an opportunity to use up the refuse from the quarry.

FIG. 86.—PIVOT PIER, GRAND FORKS BRIDGE.

For an illustrated description of the method of building concrete bridge piers, see *Engineering News*, vol. xix. pp. 443–44.

590. Specifications. The following specifications for the masonry of the railroad bridge over the Missouri River near Sibley, Mo., (Octave Chanute, engineer) may be taken as an example of the best practice.*

591. General Requirements. "The stone to be used in these piers must be of what is known as the best quality of Cottonwood limestone, or other stone which, in the opinion of the engineer, is of equally good quality and in every way suitable for the purpose for which it is to be used. It must be sound and durable, free from all drys, shakes, or flaws of any kind whatever, and must be of such a character as will, in the opinion of the engineer, withstand the action of the weather. No stone of an inferior quality will be accepted or even permitted to be delivered upon the ground. The masonry in the bridge piers must be of the best and largest stones that the quarry will afford, and must be quarried in time to season against frost before being used.

"The face stones composing the starling, and the ends and sides of the river piers from the neat line about low water up for a distance of twelve (12) feet, and also the pedestal blocks of the main piers will be of Minnesota granite, or a granite of equal quality approved by the engineer.

"All masonry of the main piers shall be regular coursed ashlar of the best description, and must be laid in mortar of the proportions of sand and cement hereinafter specified.

"All stones must be so shaped that the bearing beds shall be parallel to the natural beds, and be prepared by dressing and hammering before they are brought on the walls, as tooling and hammering will not be allowed after the stones are in place. They are to be laid to a firm bearing on their natural beds in a full bed of mortar, without the use of chips, pinners, or levelers. No shelving projections will be allowed to extend beyond the under bed on either side. The stone and work are to be kept free from all dirt that will interfere with the adhesion of mortar. Stones must be sprinkled with water before being placed in position on the wall. In laying stone in mortar, their beds are to be so prepared that when settled down they may rest close and full on the mortar. In handling the stones care must be used not to injure the joints of those already laid; and in case a stone is moved after being set and the joint broken, it must be taken out, the mortar thoroughly cleaned from the beds, and then reset.

"Wherever the engineer shall so require, stones shall have one or two 1¼-inch iron dowels passing through them and into the stones below. The holes for the dowels shall be drilled through such stones before they are put in position on the walls. After the stones are in place the holes shall be continued down into the under stones at least six (6) inches; the dowel pins will then be set in and the holes filled with neat cement grout. Clamps binding

* For specifications for first-class masonry, see § 207; see also Appendix I.

the several stones of a course together may be inserted when required by the engineer ; in such case they will be counter-sunk into the stones which they fasten together.

592. Face Stones. "The face stones must be accurately squared, jointed, and dressed on their beds and builds ; and the joints must be dressed back at least twelve inches (12) from the face. Face stones are to be brought to a joint, when laid, of not more than three quarters (¾) of an inch nor less than one half (½) inch. The courses shall not be less than eighteen (18) inches in thickness, decreasing from bottom to top of the wall. Courses to be well bonded. The face stones shall break joints at least twelve (12) inches. The face stones may be left rough, except the stones forming the starling, which must be carefully dressed to a uniform surface. The edges of face stones shall be pitched true and full to line, and on corners of all piers a chisel draft one and a half (1½) inches must be carried up from base to the under side of the coping. No projection of more than three (3) inches from the edge of face stones will be allowed. No stone with a hollow face will be allowed in the work.

593. Stretchers. "Each stretcher shall have at least twenty (20) inches width of bed for all courses of from eighteen (18) to twenty (20) inches rise, and for all thicker courses at least as much bed as rise ; and shall have an average length of at least three and one half (3½) feet, and no stretcher shall be less than three (3) feet in length.

594. Headers. "Each header shall have a width of not less than eighteen inches (18) and shall hold, back into the heart of the wall, the size that it shows on the face. The headers shall occupy at least one fifth (⅕) of the whole face of the wall, and be, as nearly as practicable, evenly distributed over it, and be so placed that the headers in each course shall divide equally, or nearly so, the spaces between the headers in the course directly below. In walls over six feet (6) in thickness, the headers shall in no case be less than three and one half feet (3½) long; and in walls over nine (9) feet thick, the headers shall be equal in length to one third the thickness of the wall, except when this length of header exceeds six (6) feet,—no header over six (6) feet long being required.

595. Backing. "The headers must alternate front and back, and their binding effect be carried through the wall by intermediate stones—not less in length and thickness than the headers of the same course—laid crosswise in the interior of the wall. The stretchers and all stones in the heart of the wall shall be of the same general dimensions and proportions as the face stones, and shall have equally good bed and bond, but may have less nice vertical joints,—although no space greater than five (5) inches in width shall be left between stones. All stones in the backing must be well fitted to their places, and carry the course evenly quite through the wall.

596. Coping. "The tops of the bridge piers, cap stones of the pedestals, and such other parts of the masonry as the engineer shall direct, shall be covered with coping of such dimensions as prescribed. All coping stones shall be neatly bush-hammer dressed on the face, bed, top, and joints; and shall be well and carefully set on the walls, brought to one quarter (¼) inch joints, and,

FIG. 87.—CHANNEL PIER, BLAIR BRIDGE.

if required, be doweled, the dowels being well secured in and to the coping with grout. No coping stone shall be less than nine (9) square feet on top.

597. Pointing. "All masonry is to be pointed so as to fill the joints solid. The surface of the wall is to be scraped clean and the joints freed of all loose mortar and refilled solid by using proper ramming tools. Joints must be well wet before being pointed. Mortar used in pointing is to be composed of one part Portland cement and one part sand.

598. Cement. "The cement used in the work shall be equal in quality to the best brands of Milwaukee or Louisville cement, and shall be ground so that at least 90 per cent. in weight will pass a standard sieve of 2,500 meshes to the square inch, and shall have a tensile strength—after being exposed one hour, or until set, in air, and the balance of the twenty-four hours in water not

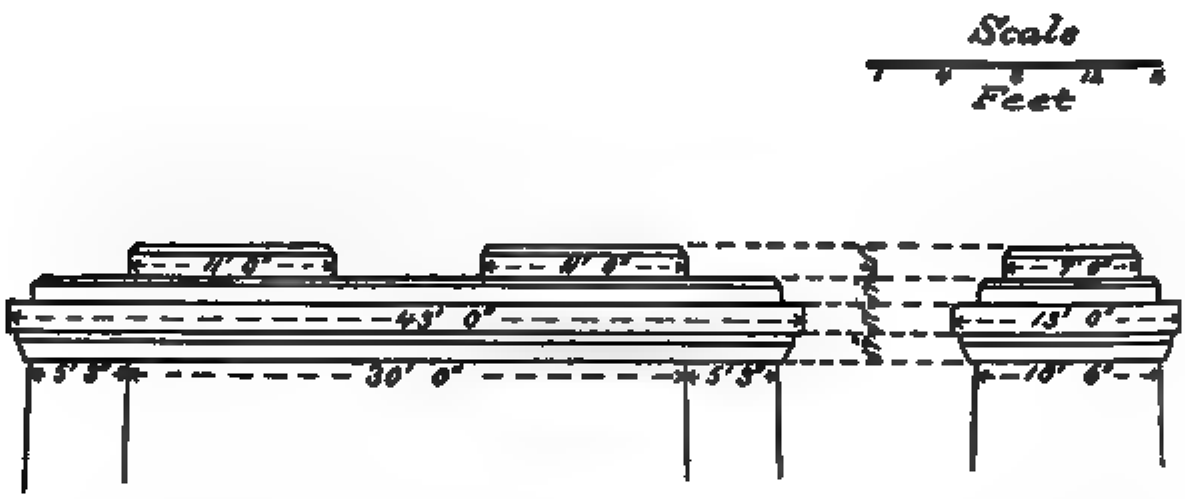

FIG. 68.—TOP OF PIER, HENDERSON BRIDGE.

below 60° F.—of at least 40 pounds per square inch; and, after being exposed one day in air and six days in water, from 60 to 100 pounds per square inch.

"All cements shall be furnished by the contractor subject to approval by the engineer. The contractor shall provide a suitable building for storing the cement, in which the same must be placed before being tested. The engineer shall be notified of the receipt of cement at least three days before it is required for use, and the inspector may take a sample from each package for testing.

599. Mortar. "The mortar shall be composed of the above cement and clean, dry, sharp sand in the proportion of one part cement to two parts of sand by weight.* The sand and cement shall be thoroughly mixed dry, and, after adding sufficient water to render the mass plastic, shall be mixed and worked until of uniform consistency throughout.

"Mortar remaining unused so long as to have taken an initial set shall not be used in the work.

* This is an unusual, but exact, method of specifying proportions; they are usually stated in volumes.

600. Pedestal Masonry. "The pedestals shall be founded upon a bed of concrete or upon piles, as may be directed by the engineer. The masonry in the pedestals shall be of the best description of coursed ashlar composed of the limestone and the mortar described above, the stones to be not less than twelve (12) inches thick, and to have horizontal beds and vertical joints on the face. When the walls do not exceed three and one half (3½) feet in thickness, the headers shall run entirely through, or a single stone—square and of the proper thickness—may be used. In walls over three and one half (3½) feet in thickness, and not over seven (7) feet in thickness, headers and stretchers shall alternate, and there shall be as many headers as stretchers. The space in the interior of the walls shall be filled with a single stone cut to fit such space, and said stone shall be of the same height as the headers and stretchers of the course. In all the masonry of these pedestals the slope must be carried up by steps and in accordance with the plans of the engineer. All the quoins must have hammer-dressed beds, builds, and joints, and draft corners."

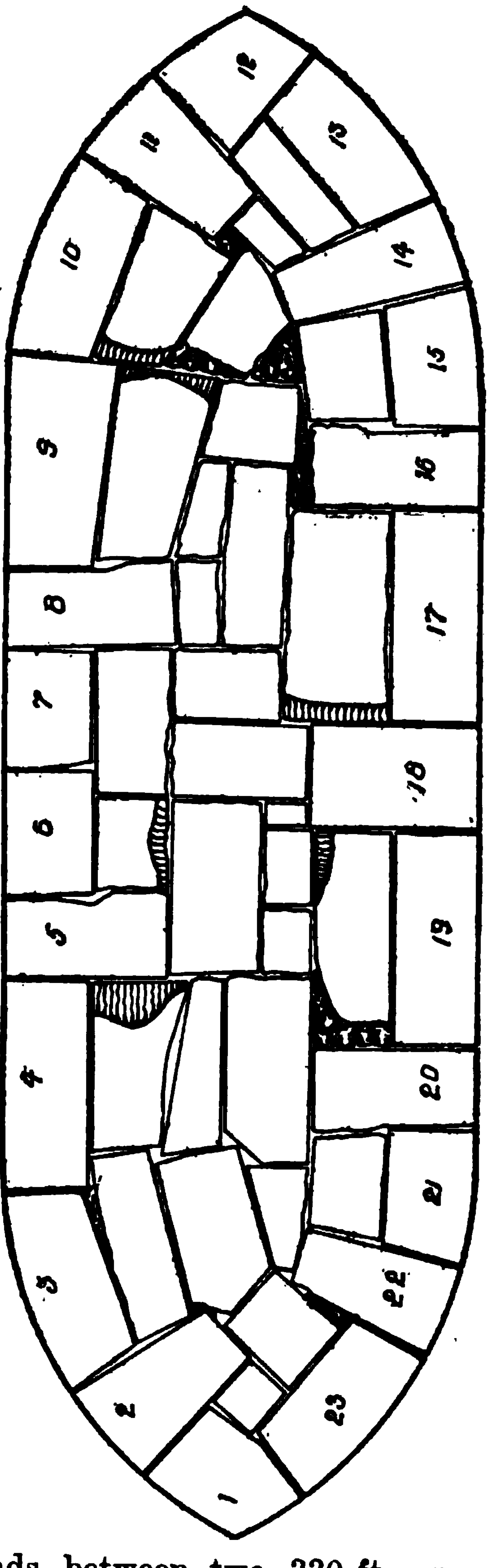

FIG. 89.—A COURSE OF A PIER OF CAIRO BRIDGE.

601. EXAMPLES OF BRIDGE PIERS. Fig. 85 (page 372) shows the channel pier of the Illinois Central R. R. bridge over the Ohio at Cairo, Ill.

Fig. 86 (page 380) shows the pivot pier of the Northern Pacific R. R. bridge over the Red River at Grand Forks, Dakota.

Fig. 87 (page 383) shows one of the two channel piers of the bridge over the Missouri River, near Blair, Neb.* This pier stands between two 330-ft. spans.

* From the Report of Geo. S. Morison, chief engineer of the bridge.

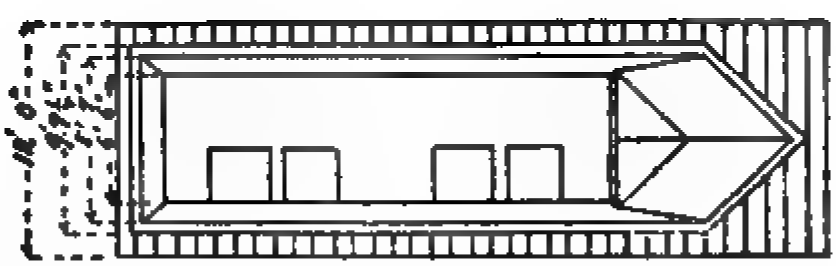

FIG. 90.—PIER OF ST. CROIX RIVER BRIDGE.

"The vertical joints are shown as they actually are in the structure." The masonry is 145 ft. from top to bottom.

Fig. 88 (page 384) shows the top of the pier between two 525-ft. channel spans of the Louisville and Nashville R. R. bridge across the Ohio River at Henderson, Ky.

Fig. 89 (page 385) shows the actual arrangement of the stones in one of the courses of one of the channel piers (Fig. 85) of the Illinois Central R. R. bridge over the Ohio River, at Cairo, Ill.

602. Fig. 90 (page 386) shows the river pier of the Chicago, Burlington and Northern R. R. bridge across the St. Croix River. This pier stands between a draw of 370 feet and a fixed span of 153 feet. The thickness of the courses is as follows, in order from the bottom up: Two courses, including the footing, 28 inches; two 26 inches; one each 24, 22, 21, 19, and 17 inches; two 15 inches; four 14 inches; one 13 inches; one 12 inches; and the coping 18 inches.

The following table gives the quantity of masonry in the pier and illustrates the manner of computing the contents of such structures. Notice that the order in the table is the same as that in the pier; *i.e.*, the top line of the table relates to the uppermost masonry, etc.

TABLE 40.

CONTENTS OF THE PIER SHOWN IN FIG. 90 (page 386).

DESCRIPTION.	DIMENSIONS.	CUBIC FEET.
Stringer Rests..	$2 \times 2.75' \times 3.0' \times 3.13'$........................	51.6
Bridge Seats....	$2 \times 2.75' \times 3.0' \times 1.46'$........................	24.1
Coping.........	$7.5' \times 24.0' \times 1.5'$........................	270.0
Neat Work.....	$\{(2 \times 6.5' + 3.6')\,23' + (2 \times 3.6' + 6.5')\,25.1'\}\frac{25.17'}{6}$..	4,579.7
" "	$(2 \times 3.6' + 7.1')\,3.8' \times \frac{18.17'}{6}$........................	279.6
Ice Breaker....	$(3.6' \times 3.6')\left(\frac{1.0'}{3} + 1.0'\right)$........................	17.3
" "	$\frac{1}{2}\,(3.6' \times 3.6' + 4.3' \times 4.3')\,18.17'$..................	285.8
Footing Course.	$9.6' \times 29.4' \times 2.33'$........................	658.5
" " .	$4.8' \times 4.8' \times 2.33'$........................	53.8
	Total = 230.39 cubic yards =	6,220.4

603. Iron Tubular Piers. For a description of an iron tubular pier, see § 415; and for a description of a pier founded upon screw piles, see *Engineering News*, vol. xiii. pp. 210–12.

604. Timber Barrel Piers. The Chicago, Burlington and Quincy R. R. has constructed a few "barrel piers" as an experiment, the object being to reduce the cost of foundations, and also to find some cheap substitute for masonry. The barrels are cylindrical, 8 feet in diameter, and 20 to 30 feet in length. The staves are 10 inches thick, 8 inches wide on the outside, and are dressed to fit together to form a cylinder. The staves are bolted at the top and bottom to two inside rings made of I-beams, and are further held in place by strong outside hoops of iron. These caissons or barrels are sunk by excavating the soil from the inside. The bottom and top portions of the caisson are filled with concrete, and the intermediate portion with sand. On top of the wooden barrel, an iron frame is placed, upon which the truss rests. Two barrels constitute a pier. The advantages claimed for the wooden caissons are that they can be put in without interfering with traffic, or without loss of time in sinking by the passage of trains. The objection to them is that they are not durable.

605. Contents of Bridge Piers. The table on page 389 gives the quantity of masonry in bridge piers having rectangular cross sections and such dimensions on top and batters as occur most frequently (see §§ 584–87). The quantities in the first four columns cover most of the cases for highway and single track railway bridges; and the quantities in the last two columns are applicable to double track railway bridges. Since that portion of the pier below the water should have more or less pointed ends, and since there is likely to be an offset in the profile—particularly of high piers,—the quantities in the table (being for a rectangular cross section) are mainly useful in making preliminary estimates.

The contents of piers of other dimensions than those in the table may be computed by the following formula: *

$$contents = t\,h\,l + b\,(l + t)\,h^2 + 1\tfrac{1}{3}\,b^2 h^3,$$

in which l = the length on top under the coping,
t = " thickness on top under the coping,
h = " height to the under side of the coping,
b = " batter—*i. e.*, $b = \frac{1}{12}$ or $\frac{1}{24}$.

The length on the bottom $= l + 2\,b\,h$; and the thickness on the bottom $= t + 2\,b\,h$. To illustrate the method of applying this for-

* See foot-note, page 358.

TABLE 41.

CONTENTS OF BRIDGE PIERS HAVING RECTANGULAR CROSS SECTION AND THE SAME BATTER ON ALL FACES.

Height—top of Footing to bottom of Coping.	Dimension of the Pier on top under the Coping.					
	5 ft. × 20 ft.		6 ft. × 22 ft.		6 ft. × 34 ft.	
	Batter 1 : 12	Batter 1 : 24	Batter 1 : 12	Batter 1 : 24	Batter 1 : 12	Batter 1 : 24
feet.	*cu. yds.*	*cu. yds.*	*cu. yds.*	*cu. yds.*	*cu. yds.*	*cu. yds.*
5	20.49	19.49	26.64	25.58	40.90	39.33
6	25.07	23.63	32.51	30.90	49.84	47.57
7	29.83	27.85	38.57	36.36	59.05	55.93
8	34.74	32.13	44.81	41.90	68.52	64.48
9	39.84	36.52	51.24	47.55	78 23	73.04
10	45.09	40.97	57.86	53.28	88.23	81.80
11	50.53	45.51	64.67	59.09	98.50	90.68
12	56.14	50.14	71.69	65.02	109.02	99.68
13	61.93	54.85	78.89	71.02	119.83	108.83
14	67.91	59.64	86.31	77.13	130.90	118.09
15	74.07	64.52	93.92	83.33	142.25	127.49
16	80.40	69.48	101.72	89.61	153.88	137.03
17	86.93	74.53	109.75	96.00	165.79	146.69
18	93.65	79.66	117.98	102.49	177.99	156.49
19	100.56	84.87	126.43	109.06	190.45	166.40
20	107.66	90.18	135.07	115.72	203.22	176.47
21	114.96	95.57	143.94	122.49	216.28	186.67
22	122.46	101.06	153.01	129.36	229.60	196.98
23	130.15	106.62	162.33	136.34	243.24	207.45
24	138.04	112.27	171.84	143.39	257.17	218.05
25	146.14	118.03	181.56	150.53	271.39	228.79
26	154.45	123.86	191.53	157.79	285.91	239.65
27	162.96	129.79	201.74	165.17	300.74	250.67
28	171.69	135.81	212.16	172.63	315.87	261.82
29	180.62	141.93	222.79	180.18	331.27	273.09
30	189.77	148.13	233.68	187.85	347.01	284.51
32	208.72	160.81	256.15	203.47	379.42	307.78
34	228.54	173.86	279.58	219.52	413.06	331.59
36	249.26	187.30	303.98	235.98	447.99	355.99
38	270.91	201.13	329.36	252.84	484.17	380.93
40	293.47	215.33	355.74	270.13	521.66	406.43
42	316.98	229.92	383.17	287.83	560.51	432.57
44	341.46	244.91	411.59	306.02	600.64	459.22
46	366.90	260.29	441.05	324.60	642.15	486.47
48	393.36	276.09	471.66	343.66	684.99	514.33
50	420.82	292.29	503.32	363.13	729.24	542.78
52	449.33	308.90	536.07	383.08	774.88	571.80
54	478.86	325.93	569.96	403.45	821.98	601.47
56	509.45	343.38	604.96	424.29	870.45	631.71
58	541.13	361.24	641.11	445.57	920.41	662.57
60	573.85	379.52	678.48	467.42	971.78	694.03

mula, assume that it is required to find the contents of a pier 4 feet thick, 20 feet long on top, and 30 feet high, having a batter on all four faces of 1 inch per foot. Then $l = 20$, $t = 4$, $b = \frac{1}{12}$, and the preceding formula becomes

$$\textit{contents} = 4 \times 20 \times 30 + \tfrac{1}{12}\,(20 + 4)\ (30)^2 + \tfrac{4}{3} \times \tfrac{1}{144} \times (30)^3$$
$$= 4{,}450 \text{ cubic feet.}$$

606. Cost. For a general discussion of the cost of masonry, see §§ 226–38 (pp. 153–60); and for data on the cost of bridge pier masonry, see § 235 (p. 157).

CHAPTER XVII.

CULVERTS.

ART. 1. WATER WAY REQUIRED.

607. The determination of the amount of water way required in any given case is a problem that does not admit of an exact mathematical solution. Although the proportioning of culverts is in a measure indeterminate, it demands an intelligent treatment. If the culvert is too small, it is liable to cause a washout, entailing possibly loss of life, interruptions of traffic, and cost of repairs. On the other hand, if the culvert is made unnecessarily large, the cost of construction is needlessly increased. Any one can make a culvert large enough; but it is the province of the engineer to design one of sufficient but not extravagant size.

608. THE FACTORS. The area of water way required depends upon (1) the rate of rain-fall, (2) the kind and condition of the soil, (3) the character and inclination of the surface, (4) the condition and inclination of the bed of the stream, (5) the shape of the area to be drained and the position of the branches of the stream, (6) the form of the mouth and the inclination of the bed of the culvert, and (7) whether it is permissible to back the water up above the culvert, thereby causing it to discharge under a head.

1. It is the maximum rate of rain-fall during the severest storms which is required in this connection. This certainly varies greatly in different sections; but there are almost no data to show what it is for any particular locality, since records generally give the amount per day, and rarely per hour, while the duration of the storm is seldom recorded. Further, probably the longer the series of observations, the larger will be the maximum rate recorded, since the heavier the storm the less frequent its occurrence; and hence a record for a short period, however complete, is of but little value in this connection. Further, the severest rain-falls are of comparatively limited extent, and hence the smaller the area, the larger the

possible maximum precipitation. Finally, the effect of the rain-fall in melting snow would have to be considered in determining the maximum amount of water for a given area.

2. The amount of water to be drained off will depend upon the permeability of the surface of the ground, which will vary greatly with the kind of soil, the degree of saturation, the condition of cultivation, the amount of vegetation, etc.

3. The rapidity with which the water will reach the water courses depends upon whether the surface is rough or smooth, steep or flat, barren or covered with vegetation, etc.

4. The rapidity with which the water will reach the culvert depends upon whether there is a well-defined and unobstructed channel, or whether the water finds its way in a broad thin sheet. If the water course is unobstructed and has a considerable inclination, the water may arrive at the culvert nearly as rapidly as it falls; but if the channel is obstructed, the water may be much longer in passing the culvert than in falling.

5. Of course, the water way depends upon the amount of area to be drained; but in many cases the shape of this area and the position of the branches of the stream are of more importance than the amount of the territory. For example, if the area is long and narrow, the water from the lower portion may pass through the culvert before that from the upper end arrives; or, on the other hand, if the upper end of the area is steeper than the lower, the water from the former may arrive simultaneously with that from the latter. Again, if the lower part of the area is better supplied with branches than the upper portion, the water from the former will be carried past the culvert before the arrival of that from the latter; or, on the other hand, if the upper portion is better supplied with branch water courses than the lower, the water from the whole area may arrive at the culvert at nearly the same time. In large areas the shape of the area and the position of the water courses are very important considerations.

6. The efficiency of a culvert may be materially increased by so arranging the upper end that the water may enter it without being retarded (see § 639). The discharging capacity of a culvert can also be increased by increasing the inclination of its bed, *provided* the channel below will allow the water to flow away freely after

having passed the culvert. The last, although very important, is frequently overlooked.

7. The discharging capacity of a culvert can be greatly increased by allowing the water to dam up above it. A culvert will discharge twice as much under a head of 4 feet as under a head of 1 foot. This can only safely be done with a well-constructed culvert.

609. FORMULAS. The determination of the values of the different factors entering into the problem is almost wholly a matter of judgment. An estimate for any one of the above factors is liable to be in error from 100 to 200 per cent., or even more, and of course any result deduced from such data must be very uncertain. Fortunately, mathematical exactness is not required by the problem nor warranted by the data. The question is not one of 10 or 20 per cent. of increase; for if a 2-foot pipe is insufficient, a 3-foot pipe will probably be the next size—an increase of 225 per cent.,—and if a 6-foot arch culvert is too small, an 8-foot will be used—an increase of 180 per cent. The real question is whether a 2-foot pipe or an 8-foot arch culvert is needed.

Numerous empirical formulas have been proposed for this and similar problems;* but at best they are all only approximate, since no formula can give accurate results with inaccurate data. The several formulas, when applied to the same problem, give very discordant results, owing (1) to the sources of error already referred to and (2) to the formulas' having been deduced for localities differing widely in the essential characteristics upon which the results depend. For example, a formula deduced for a dry climate, as India, is wholly inapplicable to a humid and swampy region, as Florida; and a formula deduced from an agricultural region is inapplicable in a city.

However, an approximate formula, if simple and easily applied, may be valuable as a nucleus about which to group the results of personal experience. Such a formula is to be employed more as a guide to the judgment than as a working rule; and its form, and also the value of the constants in it, should be changed as subsequent experience seems to indicate. With this use in view, a few formulas will be referred to briefly.

There are two classes of these formulas, one of which purports

* For a general note on empirical formulas, see § 364.

to give the quantity of water to be discharged per unit of drainage area and the other the area of the water way in terms of the area of the territory to be drained. The former gives the amount of water supposed to reach the culvert; and the area, slope, form, etc., of the culvert must be adjusted to allow this amount of water to pass. There are no reliable data by which to determine the discharging capacity of a culvert of any given form, and hence the use of the formulas of the first class adds complication without securing any compensating reliability. Most of the formulas in common use for proportioning water ways belong to this class. Such formulas will not be considered here.

The two following formulas belong to the second class.

610. Myer's Formula. Of the formulas giving a relation between the area of water way and the area to be drained, Myer's is the one most frequently used. It is

$$\textit{Area of water way, in square feet} = C\sqrt{\textit{Drainage area, in acres}},$$

in which C is a variable co-efficient to be assigned. For slightly rolling prairie, C is usually taken at 1; for hilly ground at 1.5; and for mountainous and rocky ground at 4. For most localities, at least, this formula gives too large results for small drainage areas. For example, according to the formula, a culvert having a water way of one square foot will carry the water from a single acre only. Further, if the preponderance of the testimony of the formulas for the quantity of water reaching the culvert from a given area can be relied upon, the area of water way increases more rapidly than the square root of the drainage area as required by this formula. Hence, it appears that neither the constants nor the form of this formula were correctly chosen; and, consequently, for small drainage areas it gives the area of waterway too great, and for large drainage areas too small.

611. Talbot's Formula. Prof. A. N. Talbot proposed the following formula, "more as a guide to the judgment than as a working rule:"*

$$\textit{Area of water way, in square feet} = C\sqrt[4]{(\textit{Drainage area, in acres})^3},$$

in which C is a variable co-efficient. Data from various States gave values for C as follows: "For steep and rocky ground, C varies

* Selected Papers of the Civil Engineers' Club of the University of Illinois, No. 2, pp. 14–17.

from $\frac{2}{3}$ to 1. For rolling agricultural country subject to floods at times of melting of snow, and with the length of valley three or four times its width, C is about $\frac{1}{3}$; and if the stream is longer in proportion to the area, decrease C. In districts not affected by accumulated snow, and where the length of the valley is several times the width, $\frac{1}{5}$ or $\frac{1}{6}$, or even less, may be used. C should be increased for steep side slopes, especially if the upper part of the valley has a much greater fall than the channel at the culvert."

The author has tested the above formula by numerous culverts and small bridges in a small city and also by culverts under highways in the country (all slightly rolling prairie), and finds that it agrees fairly well with the experience of fifteen to twenty years. In these tests, it was found that water ways proportioned by this formula will probably be slightly flooded, and consequently be compelled to discharge under a small head, once every four or five years.

612. In both of the preceding formulas it will be noticed that the large range of the "constant" C affords ample opportunity for the exercise of good judgment, and makes the results obtained by the formulas almost wholly a matter of opinion.

613. Practical Method. Valuable data on the proper size of any particular culvert may be obtained (1) by observing the existing openings on the same stream, (2) by measuring—preferably at time of high water—a cross section of the stream at some narrow place, and (3) by determining the height of high water as indicated by drift and the evidence of the inhabitants of the neighborhood. With these data and a careful consideration of the various matters referred to in § 608, it is possible to determine the proper area of water way with a reasonable degree of accuracy.

Ordinarily it is wise to take into account a probable increase of flow as the country becomes better improved. However, in constructing any structure, it is not wise to make it absolutely safe against every possible contingency that may arise, for the expenditure necessitated by such a course would be a ruinous and unjustifiable extravagance. Washouts can not be prevented altogether, nor their liability reduced to a minimum, without an unreasonable expenditure. It has been said—and within reasonable limits it is true—that if some of a number of culverts are not carried away

each year, they are not well designed; that is to say, it is only a question of time when a properly proportioned culvert will perish in some excessive flood. It is easy to make a culvert large enough to be safe under all circumstances, but the difference in cost between such a structure and one that would be reasonably safe would probably much more than overbalance the losses from the washing out of an occasional culvert. It is seldom justifiable to provide for all that may possibly happen in the course of fifty or one hundred years. One dollar at 5 per cent. compound interest will amount to $11.47 in 50 years and to $131.50 in 100 years. Of course, the question is not purely one of finance, but also one of safety to human life; but even then it logically follows that, unless the engineer is prepared to spend $131.50 to avoid a given danger now, he is not justified in spending $1 to avoid a similar danger 100 years hence. This phase of the problem is very important, but is foreign to the subject of this volume.

614. In the construction of a new railroad, considerations of first cost, time, and a lack of knowledge of the amount of future traffic as well as ignorance of the physical features of the country, usually require that temporary structures be first put in, to be replaced by permanent ones later. In the mean time an incidental but very important duty of the engineer is to make a careful study of the requirement of the permanent structures which will ultimately replace the temporary ones. The high-water mark of streams and the effect of floods, even in water courses ordinarily dry, should be recorded. With these data the proper proportioning of the water way of the permanent structures becomes a comparatively easy task. Upon the judgment and ability displayed in this depends most of the economical value of the improvements; for, as the road will have fixed or standard plans for culverts, abutments, piers, etc., the supervision of the construction will not be difficult.

ART. 2. BOX AND PIPE CULVERTS.

615. STONE BOX CULVERT. This culvert consists of vertical side walls of masonry with flag stones on top from one wall to the other. Masonry box culverts were constructed much more frequently formerly than at the present time. The lack of suitable stone in many parts of the West led to the adoption of vitrified pipes (§ 627) and iron pipes (§ 631) instead of masonry box culverts. However, in

many localities they are built frequently enough to warrant a brief discussion here.

616. Foundation. A common foundation for masonry box culverts is a stone pavement (§ 219) under the entire culvert, upon which the side walls rest (see Fig. 91*a*). This is not good practice; for, since the paving is liable to be washed out, it endangers the

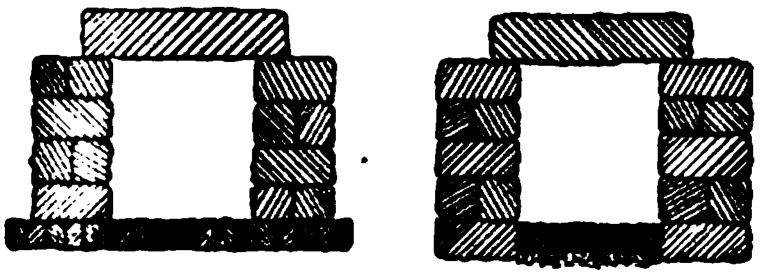

FIG. 91*a*. FIG. 91*b*.

wall. The tendency of the pavement to undermine may be diminished (1) by driving sheet piling or by setting deep curb-stones at both ends, or (2) by extending the paving to a considerable distance beyond both ends. The first is the better method; but usually these devices only postpone, and do not prevent, final failure. The water is nearly certain to carry the soil away from under the pavement, even if the curb-stones or sheet piles remain intact.

Sometimes culvert foundations are paved by laying large stones flatwise. This practice is no better than ordinary stone paving, unless the flags are large enough to extend under both walls; but stones large enough for this can seldom be obtained.

A much better method is to give each side wall an independent foundation and to pave between the walls only (see Fig. 91*b*). An important advantage of this method is that each wall can be placed separately, which facilitates the keeping of the water away from the foundation pit. Indeed, if the foundations are deep, or if there is not much current, the paving may be entirely omitted. If the current is only moderate, it is sufficient to build in, at each end of the culvert, between the ends of the side walls with solid masonry up to the bed of the stream; but if the culvert is long, it is wise to build one or more intermediate cross walls also. If the current is strong, the cross walls at the ends should be carried down deep, and the space between the side walls should be paved with large stones closely set and deeply bedded. The best job possible is secured by setting the paving in cement mortar. In this connection, see Figs. 94, 95, and 96 (pages 403, 404, and 406).

The side walls and the cross walls (particularly at the end of the culvert) should have their foundations below the effect of frost.

617. End Walls. The ends of box culverts are usually finished either with a plane wall perpendicular to the axis of the culvert as shown in Fig. 95 (page 404), or by stepping the ends off as shown in Fig. 92. Either form is liable to become clogged and to have its effectiveness greatly decreased, and probably its own existence endangered, by drift collected at its upper end. This danger is

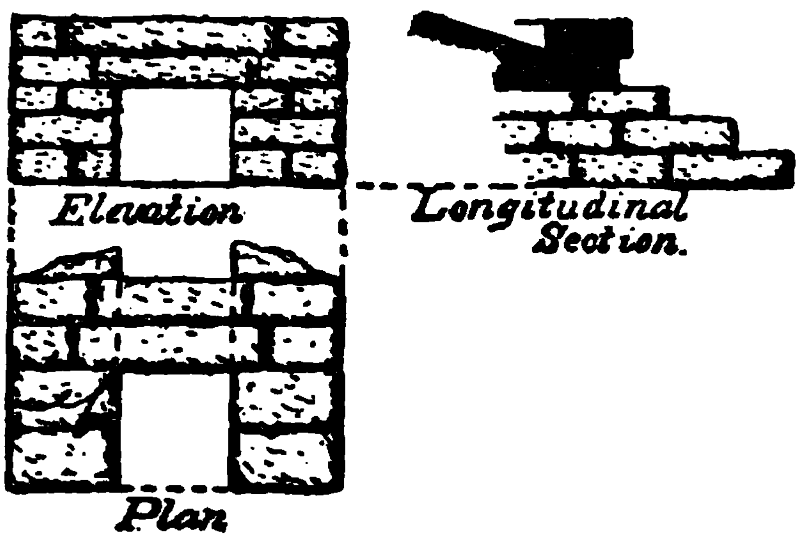

FIG. 92.

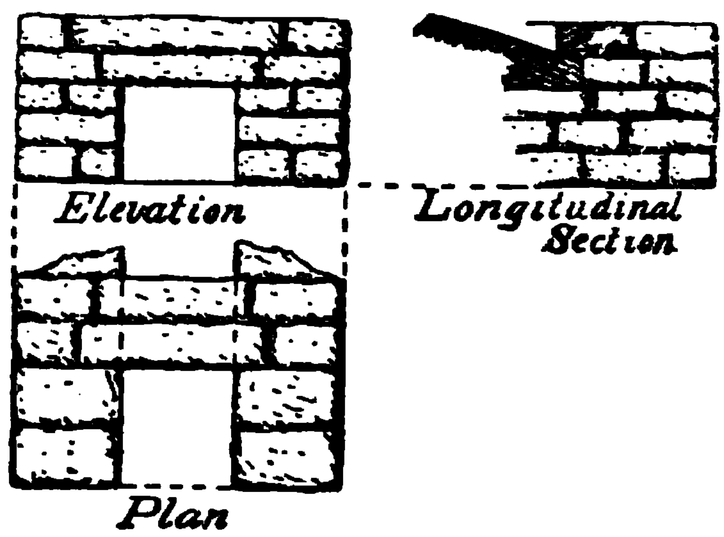

FIG. 93.

considerably decreased by extending the side walls at the upper end as shown in Fig. 93 and in Fig. 94 (page 403). If the mouth of the culvert should become stopped with drift, the open top is a well into which the water may fall. In this way the full discharging capacity of the culvert can be maintained. The lower end may be stepped as shown in Fig. 92.

The wing walls may be made thinner at the outer end, thus producing to a small degree the same effect as is obtained in splaying the wings of arch culverts (see §§ 638–39).

In this connection, see also Fig. 96 (page 406).

618. Cover Stones. To deduce a relationship between the thickness of the cover stones and the load to be supported, let

T = the thickness, *in inches;*
S = the span, *in feet;*
H = the height of bank, *in feet,* above the top of the culvert;
R = the modulus of rupture, *in pounds per square inch;*
C = the co-efficient of transverse strength (§ 15);
W = the total weight of the earth over the cover stone, *in pounds.*

For simplicity, consider a section of the culvert only a foot long. The cover stones are in the condition of a beam supported at the ends and loaded uniformly. By the principles of the resistance of materials, one eighth of the uniform load *multiplied by* the span *is equal to* one sixth of the continued product of the modulus of rupture, the breadth, and the square of the thickness. Expressing this in symbols as above, and reducing, gives

$$T = \sqrt{\frac{6}{8}\frac{W S}{R}} \qquad (1)$$

Ordinarily, earth weighs from 80 to 100 lbs. per cu. ft., but for convenience we will assume it at 100 lbs. per cu. ft., which is on the safe side; then $W = 100\,H\,S$. The maximum moving load for railroad bridges may be taken at, say, 2 tons per foot of track. This is distributed over at least 8 square feet; and hence the live load is equal to one quarter of a ton, or 500 pounds, per square foot, *i. e.* the live load is equal to an embankment 5 feet high. Therefore, the maximum live load—a locomotive—is provided for by adding 5 feet to the actual height of the embankment. The table on page 12 shows that for limestone $R = 1{,}500$. Substituting these values in equation (1), above, gives for *limestone*

$$T = 0.20\,S\,\sqrt{H + 5}, \qquad (2)$$

By substituting the corresponding value of R from the table on page 12, we have for *sandstone*

$$T = 0.25\,S\,\sqrt{H + 5}, \qquad (3)$$

For *highways*, it is sufficiently exact to drop the 5 under the radical, *i. e.*, to neglect the live load; and equation (1) then becomes for *limestone*

$$T = 0.20\,S\,\sqrt{H}, \qquad (4)$$

and for *sandstone*

$$T = 0.25\,S\,\sqrt{H}. \qquad (5)$$

The preceding formulas give the thickness which a stone of average quality must have to be on the point of breaking; and hence

in applying them it will be necessary to allow a margin for safety, either by selecting the stone or by increasing the computed thickness. If reasonable care is used in selecting the stones, it is probably safe to double the thickness found as above. To allow for any given factor of safety, multiply the thickness found by applying the above formulas by the square root of the factor of safety. Thus, to allow for a factor of 4, multiply the thickness found as above by 2; for a factor of 6, multiply by $2\frac{1}{2}$; and for a factor of 9, multiply by 3.

619. The thickness of the cover stones does not, however, depend alone upon the depth of the earth, the live load, and the span.

In the first place, the pressure on the cover stone does not vary directly as the depth of the earth above it. (*a*) The earth itself acts more or less as a beam to support part, at least, of the weight over the opening. That earth may act thus is proven by the fact that an excavation can be carried horizontally into an embankment or side hill without supporting the roof. The beam strength of the earth increases with the compactness and the tenacity of the soil and with the square of the height of the embankment above the roof. This effect would be zero with clean sand; but, owing to the nature of that material, it would seldom be employed for filling over a culvert. Hence, under ordinary conditions, part of the load is supported by the beam strength of the earth itself. Therefore, a low embankment may produce a greater strain in the cover than a much higher one. (*b*) The prism of earth directly over the culvert will be partially supported by the adjacent soil; that is to say, the particles of earth directly above the culvert will act more or less as arches resting upon the earth at the sides of the culvert, thus partially relieving the cover stones. This effect would be greater with sharp sand than with clay, but would be entirely destroyed by shock, as of passing trains. (*c*) The stones at the center of the culvert would be relieved of part of their load by an action similar to that mentioned above, whereby the weight over the center of the culvert is transferred towards its ends. However, the relief caused by this action is but slight.

In the second place, the pressure due to the live load is transmitted downward in diverging lines, thus distributing the weight over a considerably larger area than that assumed in deducing equations (2) and (3) above.

In the third place, the cover must be thick enough to resist the

effect of frost, as well as to support the earth and live load above it. The freezing, and consequent expansion, of the earth is a force tending directly to break the cover stones. That this is an important consideration is proved by the fact that these stones break near the ends of culverts as frequently as near the middle, although the weight to be supported is greater at the latter place.

620. It is impossible to compute, even approximately, the effect of the preceding factors; but experience shows that the thickness is independent of the height of the embankment, *provided* there is sufficient earth over the cover stones to prevent serious shock,—say 3 feet for railroads and 1 to 2 feet for highways.

The thickness employed on the railroads in States along the fortieth parallel of latitude is generally about as follows, irrespective of the height of the bank or of whether the cover is limestone or sandstone:

SPAN OF CULVERT.	THICKNESS OF COVER.
2 feet,	10 inches.
3 feet,	12 inches.
4 feet,	15 inches.

On the Canadian Pacific R. R., the minimum thickness of cover stones for spans of 3 feet is 16 inches, and under 3 feet, 14 inches.

621. Quality of Masonry. Box culverts are usually built of rubble masonry (§ 213) laid in cement mortar. Formerly they were often built of dry rubble, except for 3 or 4 feet at each end, which was laid in mortar. It is now generally held that box culverts should be so built that they may discharge under a head without damage. It is usually specified that the cover stones must have a solid, well-leveled bearing on the side walls of not less than 15 inches. The most careful constructors close the joints between the cover stones by bedding spalls in mortar over them.

622. *Specifications.** All stone box culverts shall have a water way at least $2\frac{1}{2} \times 3$ feet. The side walls shall not be less than two feet (2′) thick, and shall be built of sound, durable stones not less than six inches (6″) thick, laid in cement mortar [usually 1 part Rosendale cement to 2 parts sand]. The walls must be laid in true horizontal courses, but in case the thickness of the course is greater than 12 inches (12″), occasionally two stones may be used to make up the thickness. The walls must be laid so as to be thoroughly bonded, and at least one fourth of the area of each course must be headers going en-

* Pennsylvania Railroad.

tirely through the wall. The top course must have one half its area of through stones, and the remainder of this course must consist of stone going at least one half of the way across the wall from the inside face. The face stones of each course must be dressed to a straight edge, and pitched off to a true line. All of the coping stones of head walls must be throughs, and must have the upper surface hammer-dressed to a straight edge, and the face pitched off to a line with margin draft. Cover stones shall have a thickness of at least twelve inches (12″) for opening of three feet (3′), and at least 14 inches (14″) for opening of four feet (4′); and must be carefully selected, and must be of such length as to have a bearing of at least one foot (1′) on either wall.

The beds and vertical joints of the face stones for a distance of six inches (6″) from the face of the wall shall be so dressed as to require a mortar joint not thicker than three fourths of an inch (¾″). Joints between the covering stones must be not wider than three fourths of an inch (¾″), and the bearing surface of cover stones upon side walls must be so dressed as to require not more than a one-inch (1″) mortar joint.

The paving shall consist of flat stones, set on edge, at right angles with the line of the culvert, not less than twelve inches (12″) deep, and shall be laid in cement mortar and grouted.

623. Examples. The box culvert shown in Fig. 94 (page 403), is presented as being on the whole the best (see § 617). The table accompanying the diagram gives the various dimensions of, and also quantities of masonry in, box culverts for different openings. The former data and the diagrams are ample for the construction of any box culvert; while the latter data will be useful in making estimates of cost (§ 626). In the headings of the colums under "Size of the Openings," the first number is the span of the culvert, and the second is the clear height of water way. The quantities of masonry in the table were computed for a cross wall at each end of the culverts, of the section shown in Fig. 94; but in many cases, this should be 3 feet deep instead of 2, as shown. In using the table this correction is easily applied.

624. The box culvert shown in Fig. 95 is the one employed in the construction of the "West Shore R. R."—New York City to Buffalo. The data in the table accompanying the diagram give the dimensions and quantities of masonry of various sizes. In the headings under "Size of the Openings," the first number is the span of the opening and the second is its height.

Box culverts of the general form shown in Fig. 95 are sometimes built double; *i. e.*, two culverts are built side by side in such a manner as to have one side wall in common. The following table

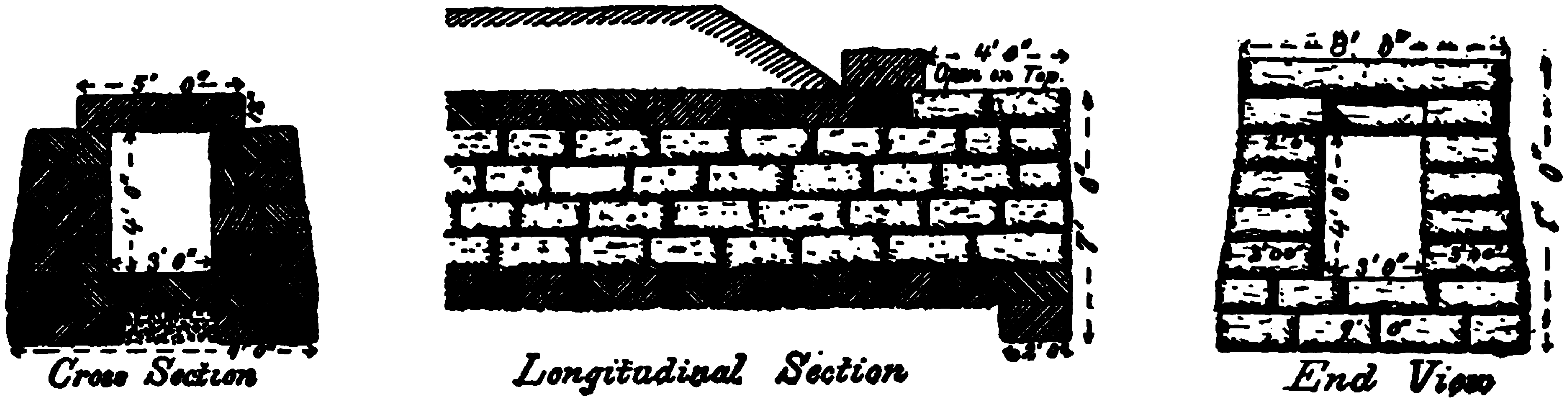

FIG. 94 STANDARD BOX CULVERT

Scale of Feet.

TABLE 42.—DIMENSIONS AND CONTENTS FOR BOX CULVERTS OF THE ABOVE GENERAL FORM.

ITEMS.	SIZE OF THE OPENING.					
	2 × 2 ft.	2 × 3 ft.	3 × 3 ft.	3 × 4 ft.	4 × 4 ft.	4 × 5 ft.
DIMENSIONS:						
Extreme width—end view	6' 0"	6' 0"	8' 0"	9' 0"	10' 0"	11' 0"
" height " "	6' 0"	7' 0"	7' 0"	8' 0"	8' 0"	9' 0"
Thickness of top of side wall	2' 0"	2' 0"	2' 6"	2' 6"	2' 6"	3' 0"
" " bottom of side wall	2' 0"	2' 0"	2' 6"	3' 0"	3' 0"	3' 6"
Length of cover stones	4' 0"	4' 0"	5' 0"	5' 0"	6' 0"	6' 0"
Thickness of cover stones and coping	10"	10"	12"	12"	12"	15"
CONTENTS:						
Masonry in two end walls—from the outer end to the back of the coping,—in cubic yards	6.86	8.04	12.80	16.88	17.44	25.68
Masonry in the trunk, per foot of distance from inside to inside of coping, in cubic yards	0.728	0.877	1.111	1.444	1.481	1.981
Paving in the trunk, per foot of distance from inside to inside of coping, in cubic yards	0.074	0.074	0.111	0.111	0.148	0.148

Cross Section. *Longitudinal Section.*

FIG. 95 BOX CULVERT—WEST SHORE R.R.

Scale of Feet.

TABLE 48.—DIMENSIONS AND CONTENTS OF BOX CULVERTS OF THE ABOVE GENERAL FORM.

Items.	Size of the Opening				
	2 × 2½ ft.	2½ × 3 ft.	[illegible] × [illegible] ft.	3 × 4 ft.	4 × 5 ft.
DIMENSIONS:					
End wall, length of	12' 6"	15' 0"	15' 0"	17' 0"	20' 0"
" " height of	7' 2"	7' 8"	8' 2"	8' 8"	9' 6"
" " thickness of	2' 0"	2' 0"	2' 6"	3' 0"	3' 6"
Side wall, "	2' 0"	2' 0"	2' 6"	3' 0"	3' 6"
Cover stones, "	10 in.	10 in.	10 in.	12 in.	15 in.
CONTENTS:					
Masonry in two end walls, complete in cubic yards	9.55	12 99	16 08	24 20	41 71
Masonry in trunk, per ft. of length from inside to inside of end walls, in cu. yds.	0.509	0 602	0 808	1 149	1 658
Paving in trunk, per foot of length from inside to inside of end walls, in cu. yds.	0 250	0 278	0 316	0.370	0 444

gives the dimensions and quantities for such box culverts. The dimensions not given in the following table are the same as in the table accompanying Fig. 95.

TABLE 44.

DIMENSIONS AND CONTENTS OF DOUBLE BOX CULVERTS.

ITEMS.	SIZE OF THE OPENING.				
	2 × 2½ feet.	2½ × 3 feet.	2½ × 3½ feet.	3 × 4 feet.	4 × 5 feet.
DIMENSIONS:					
End wall, length of	16′ 6″	20′ 0″	20′ 0″	23′ 0″	30′ 0″
Center wall, thickness of	2′ 0″	2′ 0″	2′ 0″	2′ 6″	3′ 0″
CONTENTS:					
Masonry in two end walls, in cu. yds.	13.16	16.18	21.50	33.18	53.25
Masonry in trunk, per foot of length from inside to inside of end walls, in cu. yds.	0.864	0.982	1.222	1.773	2.565
Paving in trunk, per foot of length from inside to inside of end walls, in cu. yds.	0.407	0.444	0.481	0.592	0.703

The standard double box culvert employed in the construction of the Canadian Pacific R. R. differed from the form described above in having (1) shorter end walls, and wings at an angle of 30° with the axis of the culvert, and (2) a triangular cut-water at the upper end of the division wall.

625. The culvert shown in Fig. 96 is the standard on the Intercolonial Railway of Canada, and is very substantially constructed.

626. Cost. With the data accompanying Figs. 94 and 95 (pages 403 and 404), and the table of cost of masonry on page 160, it is an easy matter to make an estimate of the cost of a box culvert. For example, assume that it is proposed to build a culvert 30 feet long—out to out of culvert proper—having a water way 3 feet wide and 4 feet high, and that estimates of the cost of the general forms shown in Fig. 94 and also of that of Fig. 95 are desired.

Estimates for a 3 × 4 ft. Box Culvert of the General Form shown in Fig. 94.

Masonry in 2 end walls—16.88 cu. yds. .... @ $3.50 per cu. yd. = $59.08
" " 25 feet of trunk (1 444 × 25 =) 36.10 cu. yds. @ $3.50 " " = 126.35
Paving " 25 " " " (0.111 × 25 =) 2.78 " " @ $2.00 " " = 5.55

Total cost $190.98

Estimates for a 3 × 4 ft. Box Culvert of the General Form shown in Fig. 95.

Masonry in 2 end walls—24.20 cu. yds. .... @ $3 50 per cu. yd. = $84.70
" " 24 feet of trunk (24 × 1.148 =) 27.55 cu. yds. @ $3.50 " " = 96.43
Paving " 24 " " " (24 × 0.370 =) 8.88 " " @ $2.00 " " = 17.76

Total cost $198.89

Transverse Section.

End Elevation.

FIG. 96.—STANDARD BOX CULVERT INTERCOLONIAL RAILWAY OF CANADA.

If the price for the masonry does not include the expense for the necessary excavation, the above estimates should be increased by the cost of excavation, which will vary with the situation of the culvert.

To make a comparison of the relative cost of the two types of culverts just mentioned, we may proceed as follows: The cost per foot of the trunk of a 3 × 4 culvert of the form shown in Fig. 94 is (1.444 cu. yds. of masonry @ $3.50 *plus* 0.111 cu. yds. of paving @ $2.00) $5.28; and the corresponding cost for Fig. 95 is (1.148 cu. yds. of masonry @ $3.50 *plus* 0.370 cu. yds. of paving @ $2.00) $4.76. The difference in cost per foot is ($5.28 – $4.76) $0.52 in favor of Fig. 95. The cost of the end walls for Fig. 94 is (16.88 cu. yds. @ $3.50) $59.08; and the corresponding cost for Fig. 95 is (24.20 cu. yds. @ $3.50) $84.70. The difference is $25.62 in favor of Fig. 94. Since in the former the cross wall extends but 2 feet below the floor of the culvert, while in the latter the end walls extend 3 feet, the difference in cost should be decreased by the cost of the difference of the foundations. If the cross walls of Fig. 94 be carried down another foot, the amount of masonry will be increased 2 cu. yds. and the cost $7.00; and the difference in cost of the end walls will be ($25.62 – $7.00) $18.62 in favor of Fig. 94. Under these conditions, for a culvert 40 feet long, the two types will cost the same; for lengths less than 40 feet Fig. 94 is the cheaper, and for lengths greater than 40 feet Fig. 95 is the cheaper. If the end walls of Fig. 95 are carried down only 2 feet, the amount of masonry will be decreased by 3.4 cu. yds. and the cost by $11.90; and then the difference of cost will be ($25.62 – $11.90) $13.72. Under this condition, for a culvert 30 feet long, the two types will cost the same; for lengths less than 30 feet Fig. 94 is the cheaper, and for lengths greater than 30 feet Fig. 95 is the cheaper. We may conclude, therefore, that for lengths under 35 or 40 feet the type shown in Fig. 94 is a little cheaper, while for greater lengths than 35 or 40 feet that in Fig. 95 is slightly cheaper. For the smallest size the length of equal cost is about 10 feet.

There is no material difference in the first cost of the two types; but the culvert shown in Fig. 94 is the more efficient.

627. Vitrified Pipe Culverts. During the past few years vitrified sewer pipes have been extensively employed for small cul-

verts under both highways and railroads. The pipe generally employed for this purpose is that known to the trade as culvert pipe or "extra heavy" or "double strength" sewer pipe, which is 20 to 40 per cent. (varying with the maker and the size) heavier than the quality ordinarily employed for sewers.

Apparently the heavier pipe is used on the supposition that the lighter is not strong enough for culverts. In most cases, at least, this is an erroneous assumption. 1. With the same depth of earth over the pipe, there is but little more pressure on the pipe when used as a culvert than when employed in a sewer. At most, the difference of pressure is that due to the live load, which can not exceed the weight of an additional 5 feet of earth (see § 618), and will generally be much less (see the second paragraph of § 619). 2. Experience demonstrates that the lighter pipes are not deficient in strength when used in sewers, however deep they are laid. According to experiments made by bedding the lower half of the pipe in sand and applying a pressure *along a comparatively narrow area,* the average crushing strength of ordinary sewer pipe was 2,400 lbs. per sq. ft. of horizontal section, and for culvert pipe 12,000 lbs. per sq. ft. If the pressure had been applied more nearly as in actual practice, the pipes would have borne considerably more. The first of the above results is equal to the weight of 24 feet of earth, and the second to that of 120 feet, although actual embankments of these heights would not give anything like the above pressures (see § 619).

There is a little difference between culverts and sewers in the exposure to frost; but no danger need be apprehended from this cause, *provided* the culverts are so constructed that the water is carried away from the lower end, since ordinary soft drain tile are not in the least injured by the expansion of the frost in the earth around them.

628. Construction. In laying the pipe, the bottom of the trench should be rounded out to fit the lower half of the body of the pipe, with proper depressions for the sockets. If the ground is soft or sandy, the earth should be rammed carefully, but solidly, in and around the lower part of the pipe. On railways, three feet of earth between the top of the pipe and the bottom of the tie has been found sufficient. On highways pipes have stood from 10 to 15 years under heavy loads with only 8 to 12 inches of earth over

them; but as a rule it is not wise to lay them with less than 12 to 18 inches of earth covering.

In many cases—perhaps in most—the joints are not calked. If this is not done, there is liability of the water's being forced out at the joints and washing away the soil from around the pipe. Even if the danger is not very imminent, the joints of the larger pipes, at least, should be calked with hydraulic cement, since the cost is very small compared with the insurance of safety thereby secured. Sometimes the joints are calked with clay. Every culvert should be built so that it can discharge water under a head without damage to itself.

The end sections should be protected with a timber or masonry bulkhead, although it is often omitted. Of course a parapet wall of rubble masonry or brick-work laid in cement is best (see Fig. 97).

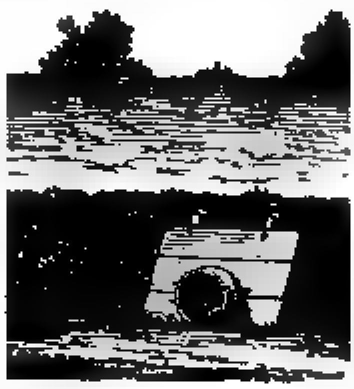

FIG. 97. FIG. 98.

The foundation of the bulkhead should be deep enough not to be disturbed by frost. In constructing the end wall, it is well to increase the fall near the outlet to allow for a possible settlement of the interior sections. When stone and brick abutments are too expensive, a fair substitute can be made by setting posts in the ground and spiking plank on as shown in Fig. 98. When planks are used, it is best to set them with considerable inclination towards the road bed to prevent their being crowded outward by the pressure of the embankment. The upper end of the culvert should be so protected that the water will not readily find its way along the outside of the pipes, in case the mouth of the culvert should become submerged.

The freezing of water in the pipe, particularly if more than half full, is liable to burst it; consequently the pipe should have a sufficient fall to drain itself, and the outlet should be so low that

there is no danger of back-water's reaching the pipe. If properly drained, there is no danger from frost.

When the capacity of one pipe is not sufficient, two or more may be laid side by side. Although two small pipes do not have as much discharging capacity as a single large one of equal cross section, yet there is an advantage in laying two small ones side by side, since then the water need not rise so high to utilize the full capacity of the two pipes as would be necessary to discharge itself through a single one of larger size.

629. Examples. Fig. 99 (page 411) shows the standard vitrified pipe culverts employed on the Kansas City and Omaha R. R. This construction gives a strong, durable culvert which passes water freely. The dimensions of the masonry end walls and of the concrete bed for the intermediate sizes are nearly proportional to those shown in Fig. 99. Table 46 (page 411) shows the quantities of masonry required for the principal sizes.

630. Cost. Prices of vitrified pipe vary greatly with the conditions of trade, and with competition and freight. Current (1888), non-competitive prices for ordinary sewer pipe, in car-load lots *f. o. b.* at the factory, are about as in the table below.

TABLE 45.

COST AND WEIGHT OF VITRIFIED SEWER PIPE.

INSIDE DIAMETER.	PRICE PER FOOT.	AREA.	WEIGHT PER FOOT.	AMOUNT IN A CAR LOAD.
12 inches.	15 cents.	.78 sq. ft.	45 lbs.	500 feet.
14 "	28 "	1.07 " "	55 "	400 "
16 "	30 "	1.40 " "	65 "	350 "
18 "	38 "	1.76 " "	75 "	300 "
20 "	53 "	2.18 " "	90 "	260 "
22 "	57 "	2.64 " "	110 "	230 "
24 "	87 "	3.14 " "	140 "	200 "

Culvert pipe costs about 20 to 25 per cent. more than as above, and second quality sewer pipe about 20 to 25 per cent. less. The latter differs from first quality in being less perfectly glazed, less perfectly burned, or not perfectly round, or in having fire cracks in the glazing, blisters on either surface, excrescences or pimples on the inside, or a piece broken out of the end. Frequently such pipe is as good for culverts as first quality sewer pipe.

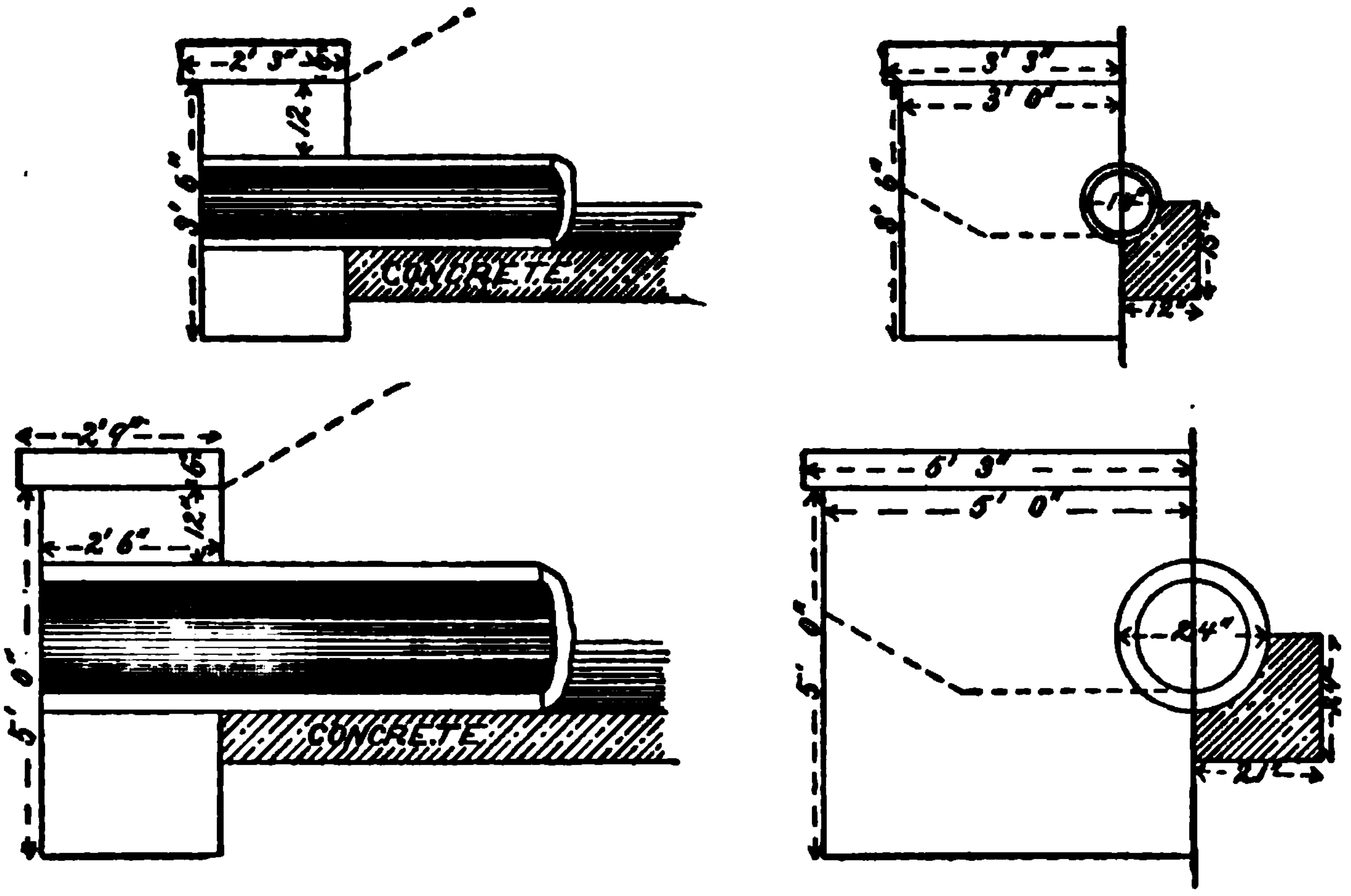

FIG. 99.—STANDARD VITRIFIED PIPE CULVERT.—K. C. & O. R. R.

TABLE 46.

MASONRY REQUIRED FOR VITRIFIED PIPE CULVERTS OF THE GENERAL FORM SHOWN ABOVE.

ITEMS.	DIAMETER OF PIPE.			
	14 inches.	16 inches.	20 inches.	24 inches.
	cu. yds.	*cu. yds.*	*cu. yds.*	*cu. yds.*
Coping, two ends..........	0.54	0.71	0.97	1.07
Parapets, two ends........	2.93	4.45	6.98	8.47
Total Masonry........	3.47	5.16	7.95	9.54
Concrete, per lineal foot..	0.070	0.102	0.136	0.180

631. Iron Pipe Culverts. In recent years, iron pipes have been much used for culverts. In many localities good stone is not available, and hence stone box culverts (§§ 615–26) can not be used. In such localities vitrified stoneware pipes are used; but as they are not made larger than 2 feet in diameter, iron or stone is the only material available for permanent culverts requiring a greater water way than that obtained by using one or two of the largest vitrified pipes. Apparently, stone culverts if well built should last forever; but, as constructed in the past, they have been found to last relatively only a short time. Hence, with the increasing cheapness of iron, there has been an increasing tendency to use iron pipe for even large culverts. Cast-iron pipes from 12 to 48 inches in diameter and 12 feet long are in common use by all of the prominent roads of the Mississippi Valley. Some of the roads cast their own, while others buy ordinary water pipe. The lightest water pipes made, or even such as have been rejected, are sufficiently strong for use in culverts. The dimensions used on the Chicago, Milwaukee and St. Paul R. R. are about as follows:

TABLE 47.

DIMENSIONS OF CAST-IRON CULVERT PIPE.

INSIDE DIAMETER.	WEIGHT PER FOOT.	THICKNESS.	WEIGHT PER LINEAL FOOT PER SQ. FT. OF AREA.
12 inches.	60 lbs.	$\frac{7}{16}$ inch.	77 lbs.
16 "	88 "	$\frac{1}{2}$ "	63 "
20 "	118 "	$\frac{5}{8}$ "	59 "
24 "	175 "	$\frac{5}{8}$ "	56 "
30 "	240 "	$\frac{3}{4}$ "	49 "
36 "	320 "	$\frac{3}{4}$ "	46 "
42 "	400 "	$\frac{7}{8}$ "	42 "
48 "	510 "	1 "	41 "

632. Construction. In constructing a culvert with cast iron, the points requiring particular attention are (1) tamping the soil tightly around the pipe to prevent the water from forming a channel along the outside, and (2) protecting the ends by suitable head walls and, when necessary, laying riprap at the lower end. The amount of masonry required for the end walls depends upon the relative width of the embankment and the number of sections of pipe used. For example, if the embankment is, say, 40 feet wide at the base, the culvert may consist of three 12-foot lengths of

pipe and a light end wall near the toe of the bank; but if the embankment is, say, 32 feet wide, the culvert may consist of two 12-foot lengths of pipe and a comparatively heavy end wall well back from the toe of the bank. The smaller sizes of pipe usually come in 12-foot lengths, but sometimes a few 6-foot lengths are included for use in adjusting the length of culvert to the width of bank. The larger sizes are generally 6 feet long.

Fig. 100 (page 414) shows the method employed on the Atchison, Topeka and Santa Fé R. R. in putting in cast-iron pipe culverts. Table 48 (page 414) gives the dimensions for the end walls for the various sizes. The length of pipe is determined by taking the multiple of 6 feet next larger than the length given by the position slope as in Fig. 100. To allow for settling, the pipe is laid to a vertical curve having a crown at the center of 1 inch for each 5 feet in vertical height from bottom of pipe to profile grade.

Where the soil is treacherous, it would be wise to lay the pipes on a bed of broken stone to prevent undue settling. In this connection, see Figs. 96 and 99 (pages 406 and 411).

633. Fig. 101 (page 415) shows the method employed on the Chicago, Burlington and Quincy R. R. of putting in cast-iron pipe culverts. This construction has given entire satisfaction.

The same road has recently commenced the use of iron for culverts up to 12 feet in diameter. For diameters greater than 4 feet, the pipes are cast in quadrants 2, 4, 6, and 8 feet long, which are afterwards bolted together, through outside flanges, to form a cylinder of any desired length. The different segments are so combined as to break joints around and also along the pipe. The body of the pipe was formerly $1\frac{3}{8}$ inches thick; but is now $1\frac{1}{8}$, stiffened on the outside by ribs. The sections are put together without any chipping, drilling, or other skilled labor. Between the different sections is a recess in which a tarred rope smeared with neat cement mortar is placed before bolting the segments together, which makes the joints tight.*

634. Cost. The cost of cast-iron pipe varies greatly with competition and the conditions of trade. The price ranges from $26 to $36 per ton for first quality water pipes, *f. o. b.* at the foundry; or approximately, say, $1\frac{1}{2}$ cents per pound.

* For illustration of details, see *Railroad Gazette*, vol. xix. pp. 122–24.

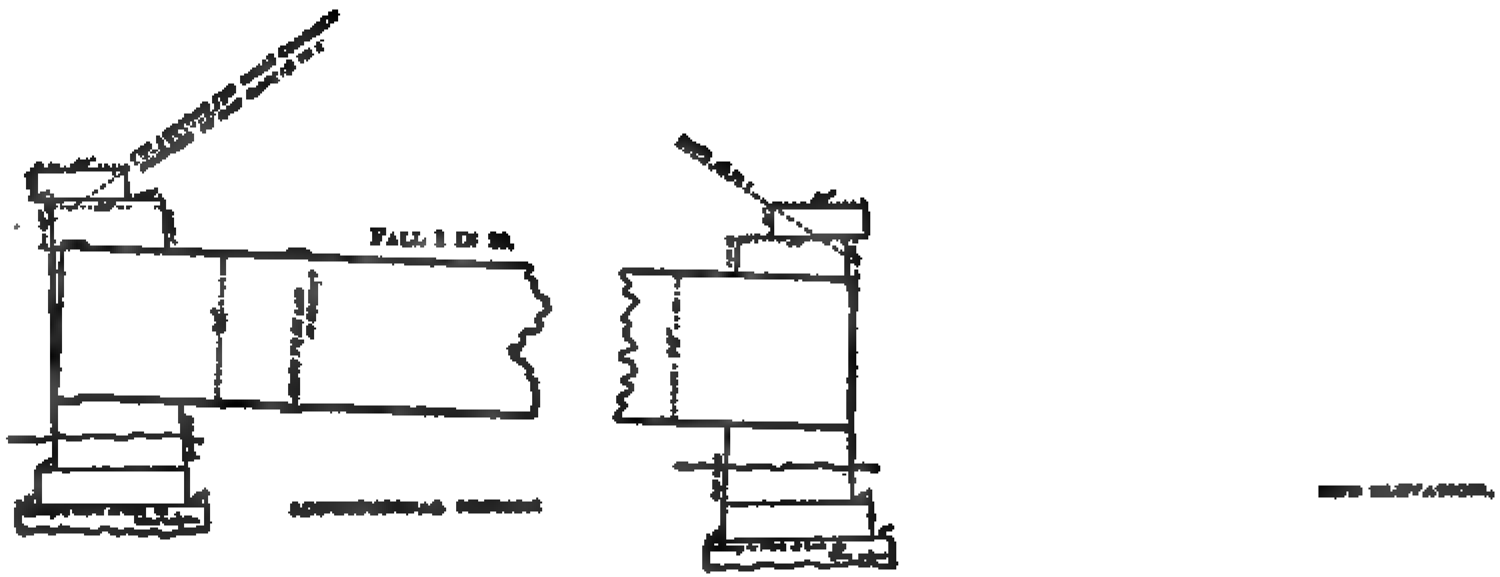

FIG. 100.—CAST-IRON PIPE CULVERTS.—A., T. & S. F. R. R. STANDARD.

TABLE 48.*

DIMENSIONS OF END WALLS FOR CAST-IRON PIPE CULVERTS SHOWN ABOVE.

Dimensions not given in the Table are the same, for all sizes, as given in Fig. 100.

Items.	Diameter of Pipe.				
	18 inches.	24 inches.	30 inches.	36 inches.	48 inches.
DIMENSIONS:					
End walls, thickness under coping	2 ft. 0 in.	2 ft. 0 in.	3 ft. 0 in.	3 ft. 0 in.	3 ft. 0 in.
" " length of	6 " 6 "	7 " 6 "	9 " 6 "	10 " 6 "	13 " 6 "
Coping, length of	7 " 6 "	8 " 6 "	10 " 6 "	11 " 6 "	14 " 6 "
Top of opening to bottom of coping	1 " 3 "	1 " 3 "	1 " 3 "	1 " 3 "	1 " 3 "
Horizontal distance marked x y on elevation	1 " 0 "	1 " 0 "	1 " 6 "	1 " 6 "	2 " 0 "

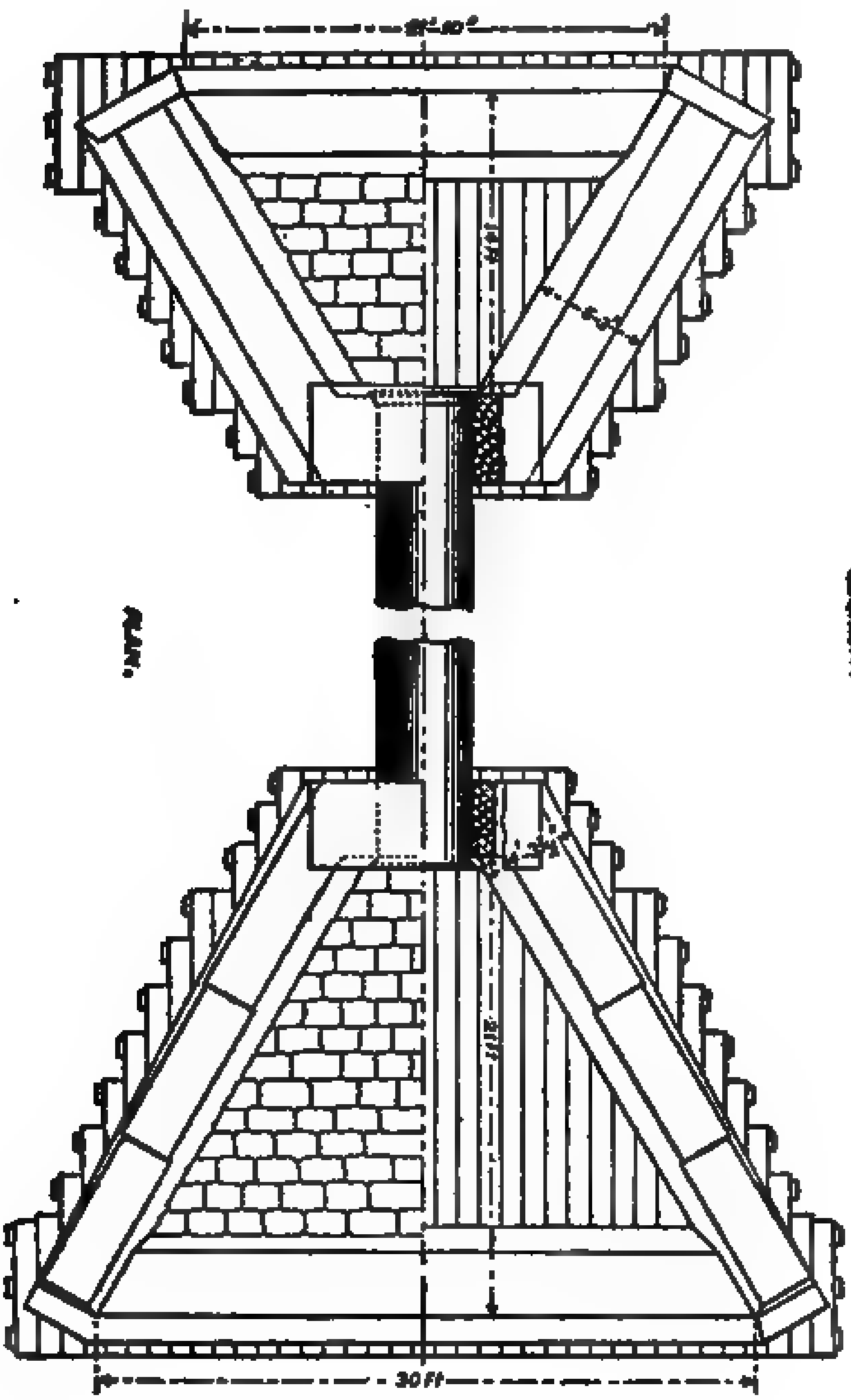

FIG. 161.—CAST-IRON PIPE CULVERT.—C., B. & Q. R. R.

Table 47 (page 412) shows that the average weight of the pipe per foot per square foot of water way is about 60 pounds; and hence the cost of the trunk of a cast-iron pipe, exclusive of transportation and labor, is about ($60 \times 1\frac{1}{2}$) 90 cents per lineal foot per sq. ft. of area. The cost of sewer pipes is, from Table 46 (page 411), about 22 cents per foot per square foot of water way; and for culvert pipe about 30 cents.

Assuming the cost of rubble masonry to be $3.50 per cubic yard and of paving to be $2.00 per cubic yard, the average cost of the masonry in the trunk of the box culvert shown in Fig. 95 (page 404) is 40 cents per lineal foot for each square foot of water way; and the corresponding cost for the culvert of Fig. 94 (page 403) is 46 cents. The end walls required for these different forms of culverts are essentially the same; and hence the above comparison shows approximately the relative cost of the different forms of culverts. According to this showing, cast-iron pipe is the most expensive; but this difference is partly neutralized by the greater ease with which the iron pipe can be put into place either in new work or in replacing a wooden box-culvert.

635. The following figures give the cost of a 7-foot cast-iron culvert of the form referred to in § 633, which see.

42 ft. body @ $26.55 per foot (1.55 cents per pound)	$1,114.83
8 ft. specials @ $29.42 " " " " " "	235.33
Bolts and washers	29.91
Unloading	17.52
Putting in place	148.95
Stone for end walls, 70 cu. yds., @ $1.50	105.00
Stone for riprap foundation, 60 cu. yds., @ $1.00	60.00
Removing temporary bridge	235.62
Total	$1,947.15

Excluding the cost of removing the temporary bridge—which is not a part of the culvert proper,—and of the riprap foundation—which the unusual conditions required,—the cost of the culvert was $33.03 per foot, or 83 cents per lineal foot for each square foot of water way.

636. TIMBER BOX CULVERTS. Timber box culverts should be used only where more substantial material is not attainable at a reasonable cost. Many culverts are constructed of timber and

periodically renewed with the same material, and many are constructed of wood and replaced with stone, or sewer or iron pipe.

The latter is an example of what may be called the standard practice in American railroad building; *i. e.*, constructing the road as quickly and cheaply as possible, using temporary structures, and completing with permanent ones later as the finances of the company will allow and as the requirements of the situation become better understood. After the line is open, the permanent structures can be built in a more leisurely manner, at appropriate seasons, and thus insure the maximum durability at a minimum cost.

There is a great variety of timber box culverts in common use, but probably there are none more durable and efficient than those used on the Chicago, Milwaukee and St. Paul R. R.,—shown in Fig. 102 (page 418).* On this road, it is the custom to replace the wooden boxes with iron pipes before the timber has seriously decayed. If experience has shown the size of the wooden box to be about right, the timbers are cut out a little and an iron pipe is placed inside of the box without disturbing the earth.

For timber box culverts of sizes larger than can be made of plank, the Atchison, Topeka and Santa Fé R. R. employs bridge-tie box culverts. These are made by laying 6 × 8 inch sawed bridge ties flatwise, in contact, to form a floor. These ties are gained at the ends so as to leave a shoulder 1 inch deep against which the inside of the side walls bears. Upon this floor, vertical side walls are constructed by laying ties flatwise, one on top of the other; the lowest timber in each side wall is fastened to each tie in the floor by a drift-bolt 12 inches long, and each timber in the side wall is fastened to the one below it by a 12-inch drift-bolt every 3 feet. The lengths of the ties employed in the side walls are so adjusted as to make the exposed ends conform closely to the slope of the embankment. The roof consists of 6- × 8-inch ties set edgewise, in close contact, with a shoulder 1 inch deep on the inside, both ends of each piece being also drift-bolted to the side wall.

637. TIMBER BARREL CULVERTS. For a number of years past the Chicago, Burlington and Quincy R. R. has found it desirable, in view of the absence or poor quality of the stone along its lines, to use a timber "barrel-culvert" when the opening is too large for a

* From *Railroad Gazette.*

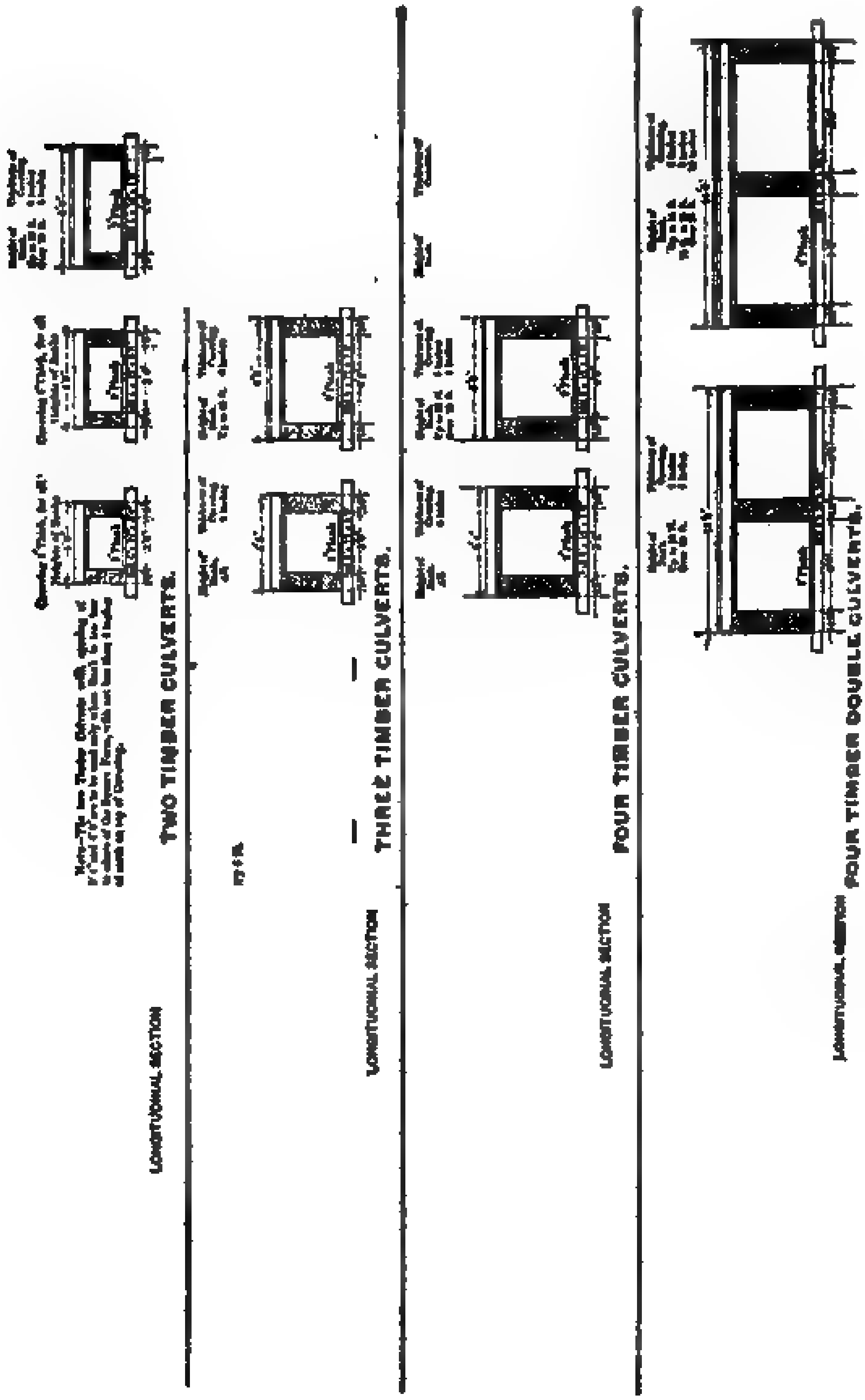

FIG. 108.—STANDARD WOOD BOX CULVERTS C., M. & ST. P. R. R.

timber box-culvert. The staves are 10 or 12 inches thick, according to the size of the culvert, and 8 inches wide on the outside, dressed to form a circle 4½ or 6 feet in diameter. Iron rings—made of old rails—spaced about 10 feet apart, are used as a form upon which to construct the culvert and also to give it strength. The staves break joints and are drift-bolted (§ 381) together. As soon as the timber is thoroughly seasoned, the culverts are lined with a single ring of brick, and concrete or stone parapet walls are built. If, at any time, the timber fails, it is the intention to put iron pipe through the present opening.

The timber costs about $12 per thousand feet, board measure, at the Mississippi River; and the cost of dressing at the company's shops is about $1.50 per thousand.

ART. 3. ARCH CULVERTS.

638. In this article will be discussed what may be called the theory of the arch culvert in contradistinction to the theory of the arch. The latter will be considered in the next chapter.

By the theory of the arch culvert is meant an exposition of the method of disposing a given quantity of masonry so as to secure (1) maximum discharging capacity, (2) minimum liability of being choked by drift, and (3) maximum strength. Attention to a few points, which are often neglected in the design of culverts, will secure these ends without additional cost.

639. GENERAL FORM OF CULVERT. Splay of Wings. There are three common ways of disposing the wing walls for finishing the ends of arch culverts. 1. The culvert is finished with a straight wall at right angles to the axis of the culvert (see Fig. 103). 2. The

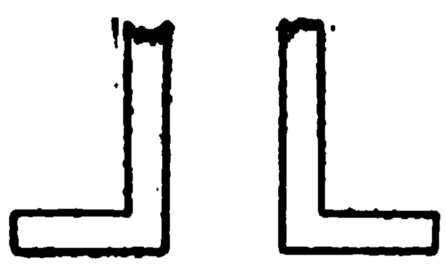

FIG. 103.

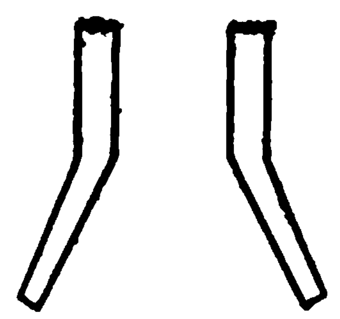

FIG. 104.

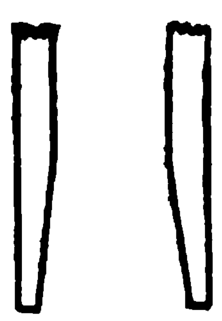

FIG. 105.

wings are placed at an angle of 30° with the axis of the culvert (see Fig. 104). 3. The wing walls are built parallel to the axis of the culvert, the back of the wing and the abutment being in a straight line and the only splay being derived from thin-

ning the wings at their outer ends (see Fig. 105). The first method is shown on a larger scale in Plate II, the third in Plate III, and the second in Plate IV.

The quantity of masonry required for these three forms of wings does not differ materially, Fig. 105 requiring the least and Fig. 103 the most. The most economical angle for the wings of Fig. 104 is about 30° with the axis.

The position of the wings shown in Fig. 104 is much the most common and is better than either of the others. Fig. 103 is objectionable for hydraulic considerations which will be considered in the next section, and also because it is more liable to become choked than either of the others. Fig. 105 does not have splay enough to admit the natural width of the stream at high water, and does not give sufficient protection to the toe of the embankment.

640. Junction of Wings and Body. With a culvert of the general form outlined in Fig. 104, there are two methods of joining the wings to the body of the culvert. The more common method is shown in Figs. 106 and 108; and the better, but less common, one is shown in Figs. 107 and 109.

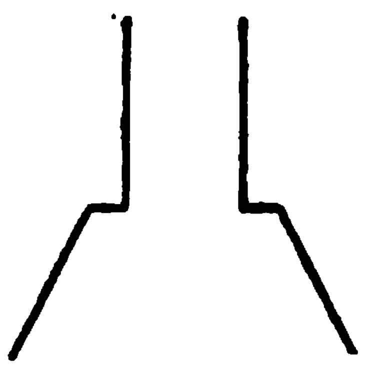

FIG. 106.

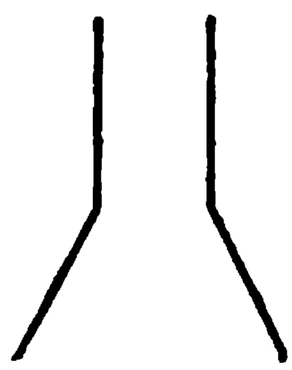

FIG. 107.

The form shown in Figs. 106 and 108 is very objectionable because (1) the corners reduce the capacity of the culvert, and (2) add to its cost.

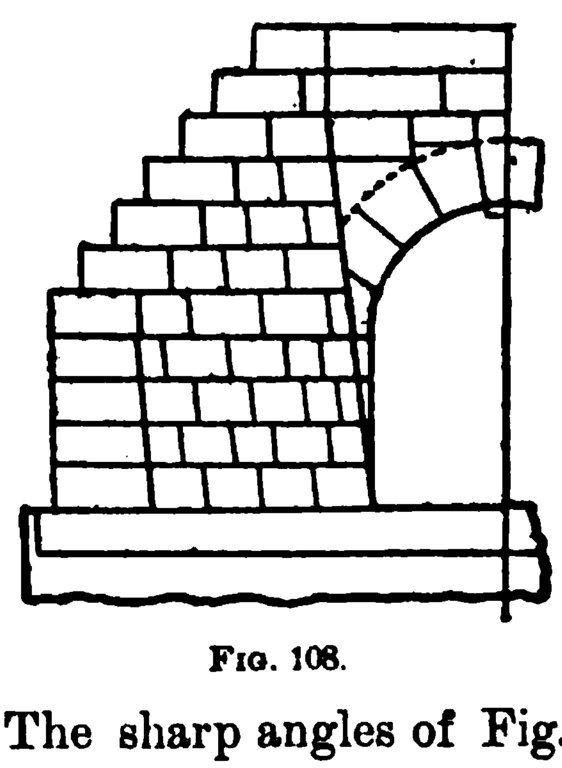

FIG. 108.

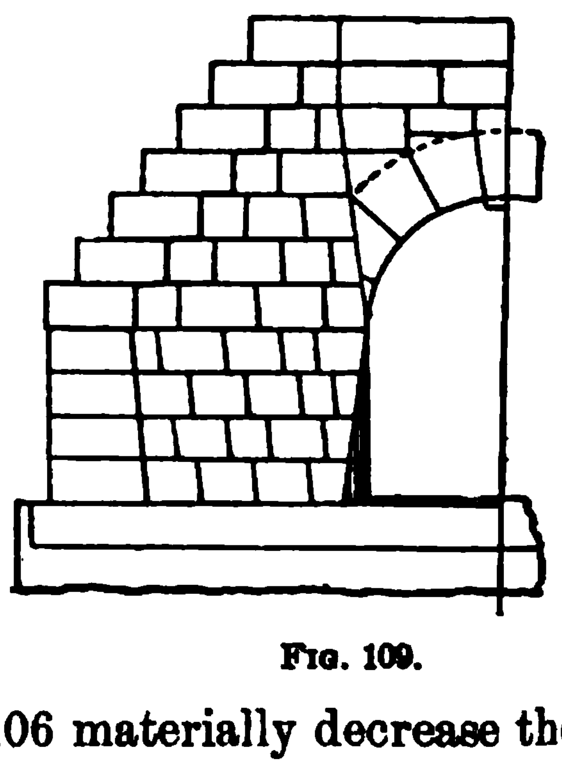

FIG. 109.

1. The sharp angles of Fig. 106 materially decrease the amount of water which can enter under a given head and also the amount

which can be discharged. It is a well-established fact in hydraulics that the discharging capacity of a pipe can be increased 200, or even 300, per cent. simply by giving the inlet and outlet forms somewhat similar to Fig. 107. Although nothing like this increase can be obtained with a culvert, one finished at both the upper and the lower end like Fig. 107 will discharge considerably more water than one like Fig. 106. The capacity of Fig. 107 decreases as the angle between the wing and the axis increases; hence, the less splay the better, provided the outer ends of the wings are far enough apart to accommodate the natural width of the stream at high water. Also the less the splay, the less the probability of the culvert's being choked with drift. Fig. 106 is very bad for both the admission and the discharge of water, and also on account of the great liability that drift and rolling stones will catch in the angles between the wings and the end walls. In this latter respect Fig. 108 is slightly better than Fig. 106.

2. Every angle adds materially to the cost of the masonry. In a culvert like Fig. 106, there are four unnecessary corners. This form probably owes its prevalence to the desire to have a uniform batter on the face of the wing, and to have the face of the wing wall intersect the end wall back of the arch stones. Satisfying both of these conditions gives a culvert in ground plan like Fig. 106; and satisfying the second one only, gives Fig. 108. Practically there is but little difference between these two forms—both are objectionable, as already explained. If the wing of Fig. 108 is moved inward, and the corner of the wing, which would otherwise project into the water way, is rounded off to a gentle curve, Fig. 109 is obtained. This form is simple, efficient, and, on the whole, the best.

Plate III shows another method of joining the wing to the end wall without having an unnecessary angle. In this case, the face of the wing up to the springing line of the arch is a warped surface, which is in some respects undesirable, although it saves a little masonry. However, the face of the wing wall could be built vertical up to the springing line and then battered; or the wing could be moved forward and the corner be rounded off as in Fig. 109.

641. Semi-circular *vs.* Segmental Arches. There are two classes of arches employed for culverts, viz., the semi-circular and

the segmental. The first is by far the more common; but nevertheless the latter is, on the whole, much the better.

1. For the same span, the segmental arch requires a shorter intrados (the inside curve of a section of the arch perpendicular to its axis). For example, the culverts shown in Plates IV and V have the same span, but the intrados of the semi-circular arch is 15.71 ft., while that of the segmental arch is 10.72 ft.; that is, the intrados of the segmental is only 68 per cent. of the intrados of the semi-circular arch. This difference depends upon the degree of flatness of the segmental arch. The above example is an extreme case, since the segmental arch is unusually flat, the central angle being only 73° 44′. (The rise is one sixth of the span.) With a central angle of 120°, the intrados of the segment is 77 per cent. of the semi-circle.

Or, to state the above comparison in another and better form, for the same length of intrados the segmental arch gives the greater span. For example, a segmental arch on the same general plan as that of Plate V, but having an intrados equal to that of Plate IV, would have a span of 14.64 ft., which is 46 per cent. greater than the span of the semi-circular arch shown in Plate IV. A segmental arch with a central angle of 120° has a span 33 per cent. greater than a semi-circular arch having the same length of intrados. This difference constitutes an important advantage in favor of the segmental arch culvert, since the wider the span the less the danger of the culvert's being choked by obstructions, and because it will pass considerably more water for the same depth.

2. For the same length of intrados, the segmental arch gives the greater water way. The water way of the culvert shown in Plate IV is 87.6 square feet; but the same length of intrados in a segmental arch culvert having 73° 44′ central angle (the same as Plate V) would have a water way of 98.3 square feet; and with a central angle of 120° would have a water way of 99.5 square feet. In both examples the increase is one eighth.

3. On the other hand, the segmental culvert will require a thicker arch. It will be shown in the next chapter that arches can not be proportioned strictly in accordance with mathematical formulas; and hence the exact difference in thickness of arch which should exist between a semi-circular and a segmental arch can not be computed. According to established rules of practice, small

segmental arches are from 10 to 25 per cent. thicker than semi-circular ones. This difference is not very great, and its effect upon the cost of the culvert is, proportionally, still less, since the cost per yard of arch masonry is less for the thicker arches. Then, we may conclude that, since for the same span the intrados of segmental arches is from 20 to 40 per cent. shorter than the semi-circle, the segmental arch requires a less volume of arch masonry than the semi-circular, and also costs less per cubic yard. The arch masonry per foot of length of the segmental arch culvert shown in Plate V is only 71 per cent. of that in the semi-circular one shown in Plate IV. The dimensions and contents of arch culverts of the general forms shown in Plates IV and V are given in Tables 51 and 52 (pp. 430 and 431 respectively), from which it appears that the segmental arch contains only from 60 to 76 per cent. as much masonry as the semi-circular, the average for the six spans being almost exactly 70 per cent. The cost of these two classes is shown in Tables 56 and 57 (pages 437 and 438), from which it appears that the average cost of segmental culverts 20 feet long and of different spans is only 59 per cent. of the cost of semi-circular ones of the same length and span; and the average cost of an additional foot in length for the segmental is only 86 per cent. of that for a circular one. The water ways of the semi-circular culverts are a little the greater, and hence the difference in cost per square foot of water way is not as great as above; but, on the other hand, the form of water way of the segmental culvert is the more efficient, and hence the above comparison is about correct.

4. Will the segmental, *i. e.*, the flatter, arch require heavier abutments (side walls)? Unquestionably the flatter the arch the greater the thrust upon the abutment; but the abutment not only resists the thrust of the arch which tends to turn it over outwards, but also the thrust of the embankment, which tends to push it inwards. It is impossible to compute, with any degree of accuracy, either the thrust of the arch or of the embankment; and hence it is impossible to determine either the relative value of these forces or the thickness which the two abutments should have. Experience seems to indicate that the thrust of the earth is greater than that of the arch, as is shown by the fact that nearly all semi-circular culverts have abutments of much greater thickness than are required to resist the thrust of the arch; and hence we may conclude that expe-

rience has shown that the thrust of the earth necessitates a heavier abutment than does the thrust of the arch. If this be true, then the abutment for segmental arches may be thinner than those for semi-circular ones; for, since the thrust of the former is greater than the latter, it exerts a greater force outward, which counterbalances a larger part of the inward thrust of the embankment, and thus leaves a less proportion of the latter to be resisted by the mass of the abutment. Segmental arch culverts are not often built; and designers appear to have overlooked the thrust of the earth, since the side walls of segmental arches are generally thicker than for semi-circular ones (compare Plates IV and V).

The conclusions may, therefore, be drawn that segmental arch culverts are both cheaper and more efficient than semi-circular ones.

642. As built, many semi-circular arches are practically segmental; that is, the side walls are built so high, or the backing is made so heavy, that practically the abutments are less than 120° apart, and hence the two lower ends of the arch are really only a part of the side wall, and should be built square.

Further, it is shown in §§ 681–82 that a true arch of more than about 90 to 120 degrees is impossible.

643. Examples. Under this head will be given a brief description of four series of arch culverts which are believed to be representative of the best practice.

644. Illinois Central Arch Culverts. Plate II shows the general plan of the standard arch culvert employed in the construction (1852–53) of the Chicago branch of the Illinois Central Railroad.* While the timber in the foundation is apparently still in good condition, the use of timber for such shallow foundations can not be considered as the best construction. However, many of the conditions, particularly drainage, have greatly changed since this road was built, and it is by no means certain that this use of timber was not good practice at that time (see § 636).

Table 49 (page 425) gives the dimensions and contents for the several spans of this form of culvert. The contents of the end walls were computed on the assumption that the off-set at the back was 6 inches for each foot, counting from the top, until the full thickness at the bottom was obtained (see Section E–F, Plate II).

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TABLE 49.—DIMENSIONS AND CONTENTS OF ILLINOIS CENTRAL ARCH CULVERTS.

FOR DIAGRAM SEE PLATE II.

Dimensions not given in the table are the same, for all sizes, as in the diagram.

ITEMS.	SPAN. 6 ft.		8 ft.		10 ft.		12 ft.	
DIMENSIONS:								
End walls, length	22 ft.	0 in.	26 ft.	0 in.	30 ft.	0 in.	34 ft.	0 in.
height—bottom of footing to bottom of coping	6 "	9 "	7 "	9 "	9 "	0 "	10 "	0 "
thickness under coping	2 "	0 "	2 "	6 "	2 "	6 "	2 "	6 "
" top of footing	3 "	6 "	4 "	0 "	4 "	0 "	4 "	6 "
Arch, thickness at crown	1 "	3 "	1 "	3 "	1 "	6 "	1 "	9 "
" " springing	2 "	6 "	2 "	6 "	2 "	6 "	3 "	0 "
Side walls, f footing	2 "	6 "	2 "	6 "	2 "	6 "	3 "	0 "
of footing	1 "	0 "	1 "	0 "	1 "	0 "	1 "	0 "
Platform, f arch	24 "	0 "	28 "	0 "	32 "	0 "	36 "	0 "
" "	6 "	10 "	7 "	4 "	7 "	4 "	7 "	10 "
" " under trunk	14 "	0 "	16 "	0 "	18 "	0 "	21 "	0 "
CONTENTS:								
Two end walls—to back of arch and including one footing course	20.77	cu. yds.	32.97	cu. yds.	46.88	cu. yds.	59.08	cu. yds.
Arch, full bedded, per foot of length—out to out of end walls	0.537 "	"	0.678 "	"	1.008 "	"	1.399 "	"
backing, per foot of length—out to out of end walls	0.230 "	"	0.284 "	"	0.273 "	"	0.415 "	"
Two side walls, including footing, per foot of length—out to out of end walls	0.444 "	"	0.444 "	"	0.444 "	"	0.518 "	"
Coping, on two end walls	55.0 cu. ft.		65.0 cu. ft.		75.0 cu. ft.		85.0 cu. ft.	
Timber, area to be covered w ıdation	164 sq. ft.		205 sq. ft.		235 sq. ft.		282 sq. ft.	
area to be covered ounda- tion—	7 " "		8 " "		9 " "		10½ " "	
Planking, under end walls	1,282 ft. B. M.		1,498 ft. B. M.		1,707 ft. B. M.		1,992 ft. B. M.	
under trunk, per foot of length—inside to inside of wings of foundation	26 " "		32 " "		36 " "		42 " "	

645. *Example of the Use of Table* 49. To illustrate the method of using the above table, assume that an estimate of the amount of material in an 8-ft. arch culvert of the preceding form is required. Assume that the top of the coping is 3 feet below sub-grade, *i. e.*, that there is 4.25 feet of earth above the crown of the arch. Assume also that the road bed is 16 feet, and that the slope of the embankment is 1½ to 1. Then the length of the culvert from inside to inside of the end walls will be $16 + 2\ (\frac{3}{2} \times 3) = 16 + 9 = 25$ feet; and from out to out of end walls, the length will be $25 + 2 \times 2.5 = 30$ feet.

Assuming that the timbers under the planking are 8×10 inches, the 205½ sq. ft., as per the table, will require 1,639 ft. B. M. of timber, or 9 pieces 24 ft. long. Notice, however, that in practice 10 pieces would be used—5 at each end of the culvert. The length of the trunk of the foundation is $30 - 2\ (4 + \frac{1}{2} + 1) = 19$ ft. Hence the area under the trunk of the foundation to be covered with timber is 19×8 (see table) $= 152$ sq. ft.; and if 8×10-inch timbers are used, this will require 1,216 ft. B. M., or 12 pieces 14 feet long. The plank under the wings and in the sheet piling is 1,493 feet (see table), and that in the trunk is 32 (see table) $\times 19 = 608$ ft. B. M.; hence the total plank is $1{,}493 + 608 = 2{,}101$ ft. B. M.

The masonry in the end wall is 32.97 cu. yds., as in table. The masonry in 1 foot of arch is (see table) $0.673 + 0.284 = 0.957$; and in 30 ft. it is $0.957 \times 30 = 28.71$ cu. yds. The masonry in the side walls (abutments of the arch) is 0.444 (see table) $\times 30 = 13.32$ cu. yds. The coping is 65 cu. ft. (see table) $= 2.41$ cu. yds.

Collecting and tabulating the preceding results, we have the following:

Timber:—	10 pieces, 8 × 10 inches, 24 ft. long......	1,866 ft. B. M.
	12 " " " 14 " "	1,280 " "
	2-inch plank...........................	2,101 " "
	Total timber in culvert 25 ft. long....	5,247 " "
Masonry:—	2 end walls............................	33.0 cu. yds.
	coping................................	2.4 " "
	side walls (abutments).................	13.3 " "
	arch masonry..........................	28.7 " "
	Total masonry in culvert 25 ft. long..	77.4 " "

646. Chicago, Kansas and Nebraska Arch Culverts.—The culvert shown in Plate III is the standard form employed on the Chicago, Kansas and Nebraska Railroad.* Notice that the slope line intersects the inside face of the end wall at a considerable distance above the back of the crown of the arch (see Side View, Plate III). This is sometimes urged as an objection to this form of construction, on account of the supposed liability of the top of the end wall being pushed outward; but there is no danger of this method of failure, since the height of the end wall above the crown of the arch is, exclusive of the coping, only equal to its thickness, and in addition it is buttressed on the outside by the wings. The advantage of this construction is that it requires less masonry and also less labor.

Concerning the manner of joining the wings to the body, see the last paragraph of § 640 (page 421).

Table 50 (page 428) gives the dimensions and contents for various spans. The contents of the wings above the springing line of the arch were computed for courses 1 foot thick and for an earth slope of $1\frac{1}{2}$ to 1 (see § 557).

647. *Example of the Use of Table* 50. Assume the same depth of earth over the crown of the arch as in the example in § 644, *i. e.*, 4.25 ft.; and assume also that the slope line strikes the upper corner of the coping instead of the lower as shown in Plate III. The top of the coping will be 0.75 ft. below sub-grade; and, for a 16-ft. road-bed, the length of the arch—inside to inside of end walls—is $16 + 2(\frac{3}{2} \times 0.75) = 18.25$ ft. With the above data and Table 50, we have the following:

Four wing walls, including one footing course, . .	40.5 cu. yds.
Two head " " " " " . .	36.8 " "
Coping, ...	1.8 " "
Two side walls, $18\frac{1}{4}$ ft. @ 1.382 cu. yds. per foot, . .	25.2 " "
Arch masonry, " " " 1.184 " " " " . .	21.6 " "
Paving, 23.58 ft. @ 0.272 cu. yd. per ft., ...	6.4 " "
Total masonry in culvert $18\frac{1}{4}$ ft. long, . .	132.3 " "

In attempting to make comparisons between the above total and that of § 645, notice that the culverts are of very different style (see §§ 638 and 639) and that the water ways are of different areas.

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TABLE 50.

Dimensions and Contents of Chicago, Kansas and Nebraska Arch Culverts.

FOR DIAGRAM SEE PLATE III.

Dimensions not given in the table are the same, for all sizes, as in the diagram.

Items.	Designated in Diagram by	Span. 3 ft.	4 ft.	6 ft.	8 ft.	10 ft.
Dimensions:						
End wall, height above the footing	A B	6 ft. 0 in.	9 ft. 0 in.	11 ft. 0 in.	12 ft. 0 in.	13 ft. 0 in.
" length	B C	6 " 4 "	10 " 0 "	14 " 0 "	16 " 0 "	18 " 0 "
" thickness		2 " 8 "	2 " 8 "	2 " 8 "	2 " 8 "	2 " 6 "
Coping on end wall, width		3 " 0 "	3 " 0 "	3 " 0 "	3 " 0 "	3 " 6 "
" " " " length		9 " 0 "	10 " 8 "	14 " 6 "	16 " 8 "	18 " 6 "
Wing, height at inner end	X Y	4 " 3 "	7 " 8 "	9 " 3 "	10 " 1 "	11 " 8 "
" " outer "	V W	2 " 0 "	3 " 0 "	4 " 0 "	4 " 0 "	4 " 0 "
length for slope of 1½ to 1	X V	4 " 10 "	7 " 10 "	9 " 4 "	10 " 10 "	12 " 4 "
thickness on top	J K	3 " 0 "	2 " 8 "	2 " 8 "	2 " 6 "	2 " 8 "
" at bottom of outer end		2 " 2 "	2 " 6 "	3 " 0 "	3 " 0 "	3 " 0 "
distance between outer ends of wings	N O	4 " 0 "	5 " 0 "	6 " 0 "	10 " 0 "	12 " 0 "
Arch, height of springing above footing	H Z	2 " 0 "	3 " 0 "	4 " 0 "	4 " 0 "	4 " 9 "
thickness at crown	D E	0 " 8 "	1 " 0 "	1 " 8 "	1 " 6 "	1 " 9 "
Side walls, thickness at springing	R S	2 " 0 "	2 " 6 "	2 " 8 "	3 " 0 "	3 " 6 "
" top of footing	T U	2 " 8 "	3 " 0 "	4 " 0 "	4 " 0 "	4 " 6 "
Paving, depth		1 " 6 "	1 " 0 "	1 " 0 "	1 " 0 "	1 " 0 "
Contents:						
one footing course		7.7 cu. yds.	19.7 cu. yds.	25.5 cu. yds.	40.5 cu. yds.	53.4 cu. yds.
" " "		10.6 " "	16.2 " "	20.1 " "	36.6 " "	44.3 " "
one footing course—per e to inside of end walls		0.599 " "	0.888 " "	1.338 " "	1.362 " "	1.567 " "
of length from inside to [illegible]		0.270 " "	0.478 " "	0.791 " "	1.184 " "	1.719 " "
Paving, per ft. of length from out to out of end walls		0.086 " "	0.158 " "	0.198 " "	0.279 " "	0.346 " "
Coping, on two end walls		27.0 cu. ft.	32.0 cu. ft.	44.0 cu. ft.	60.0 cu. ft.	66.0 cu. ft.

648. Atchison, Topeka and Santa Fé Arch Culverts. Plates IV and V show the standard semi-circular and segmental arch culverts used by the Atchison, Topeka and Santa Fé Railroad.*

Tables 51 and 52 give the dimensions and contents for the several spans. Notice that the heights of the end walls do not vary uniformly, that for the 12-foot span being proportionately too great; and consequently the contents of the end walls and of the wings do not vary uniformly. The contents of the facing of the wings were computed for courses 18 inches thick (see § 557), and the backing was computed on the assumption that the back surface was a plane such that the dimension at the outer end and also where a plane parallel to the section E–F passes through the corner of the end wall is as in the diagram.

In computing the masonry in a given culvert, these tables are to be employed as already explained for Tables 49 and 50—see §§ 645 and 647.

649. Standard Arch Culvert. The culvert shown in Plate VI has been designed in accordance with the principles laid down in the preceding discussion (§§ 638–41). The wings are joined to the body in such a manner as to offer the least possible resistance to the passage of water and drift. If the current is slow and not liable to scour, the paving may be omitted, since the end walls, being continuous under the ends of the water way, will prevent undermining of the side walls; or, in long culverts, one or more intermediate cross walls may be constructed. But ordinarily the money paid for paving is a good investment. If the current is very rapid, it is wise to grout the paving,—and also to inspect the structure frequently.

The arch ring is amply strong to support any bank of earth (see Table 63, page 502, particularly Nos. 9, 12, 18, 53, 54, and 61). The strains in a masonry arch can not be computed exactly; but the best method of analysis (§ 688) shows that if the earth is 10 feet thick over the crown, the maximum pressure is not more than 55 pounds per square inch (compare with § 222 and also §§ 246–48). A greater thickness of earth at the crown would doubtless increase the maximum pressure in the arch; but proportionally the pressure would increase much less rapidly than the height of the bank (see

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TABLE 51.

DIMENSIONS AND CONTENTS OF ATCHISON, TOPEKA AND SANTA FÉ SEMI-CIRCULAR ARCH CULVERTS.

FOR DIAGRAM SEE PLATE IV.

Dimensions not given in the table are the same, for all sizes, as in the diagram.

ITEMS.	SPAN. 6 ft.	8 ft.	10 ft.	12 ft.	14 ft.	16 ft.
DIMENSIONS:						
End wall, length	24 ft. 10 in.	17 ft. 7½ in.	19 ft. 7½ in.	21 ft. 7½ in.	23 ft. 7½ in.	25 ft. 7½ in.
height—top of footing to bottom of coping	9 " 11 "	12 " 8⅝ "	14 " 2¼ "	14 " 8¼ "	16 " 1⅝ "	18 " 3 11/16 "
Coping, length	25 " 10 "	18 " 7½ "	20 " 7½ "	22 " 7½ "	24 " 7½ "	26 " 7½ "
Arch, thickness at crown	0 " 10 "	0 " 11 "	1 " 0 "	1 " 1 "	1 " 2 "	1 " 2 "
" at springing	3 " 0 "	3 " 0 "	3 " 6 "	4 " 0 "	4 " 0 "	4 " 6 "
number of voussoirs (arch stones)	9	11	15	17	21	23
width of voussoirs at intrados	1 " 9/16 "	1 " 1 11/16 "	1 " 9/16 "	1 " 1 5/16 "	1 " 9/16 "	1 " 1⅝ "
Side wall, thickness at springing	3 " 0 "	3 " 0 "	3 " 6 "	4 " 0 "	4 " 0 "	4 " 6 "
" " top of footing	3 " 0 "	3 " 6 "	4 " 0 "	4 " 0 "	4 " 6 "	5 " 0 "
height of, above top of footing	3 " 6 "	5 " 0 "	6 " 0 "	6 " 0 "	7 " 0 "	8 " 0 "
Wing, length for slope of 1½ to 1	none	23 " 2 "	25 " 9 "	26 " 7 "	29 " 1 "	32 " 11 "
thickness at bottom of section E–F	none	4 " 6 "	5 " 0 "	5 " 6 "	6 " 6 "	7 " 0 "
CONTENTS:						
Two end walls, above footing	30.92 cu. yds.	22.95 cu. yds.	28.53 cu. yds.	28.17 cu. yds.	30.12 cu. yds.	35.64 cu. yds.
Arch, per foot of length—inside to inside of end walls	0.765 " "	1.072 " "	1.469 " "	1.935 " "	2.274 " "	2.807 " "
Two side walls, per foot of length—inside to inside of end walls	0.777 " "	1.213 " "	1.666 " "	1.777 " "	2.140 " "	2.783 " "
Four wings, facing—slope 1½ to 1	none	46.00 " "	56.88 " "	60.74 " "	72.35 " "	99.14 " "
backing " " " "	none	19.02 " "	28.21 " "	34.78 " "	52.05 " "	73.76 " "
Paving, per foot of length—out to out of end walls	0.260 cu. yds.	0.346 " "	0.432 " "	0.518 " "	0.605 " "	0.691 " "
Coping, on two end walls	103.33 cu. ft.	74.50 cu. ft.	82.50 cu. ft.	90.50 cu. ft.	98.50 cu. ft.	106.50 cu. ft.

TABLE 53.

DIMENSIONS AND CONTENTS OF ATCHISON, TOPEKA AND SANTA FÉ SEGMENTAL ARCH CULVERTS.

FOR DIAGRAM SEE PLATE V.

Dimensions not given in the table are the same, for all sizes, as in the diagram.

ITEMS.	SPAN. 6 ft.	8 ft.	10 ft.	12 ft.	14 ft.	16 ft.
DIMENSIONS:						
End wall, length	18 ft. 2½ in.	18 ft. 7½ in.	20 ft. 7½ in.	22 ft. 7½ in.	24 ft. 7½ in.	26 ft. 7½ in.
height—top of footing to bottom of coping	6 " 1½ "	8 " ¾ "	9 " 2½ "	9 " 6¼ "	9 " 4 "	9 " 8 "
Coping, length	19 " 2½ "	19 " 7½ "	21 " 7½ "	23 " 7½ "	25 " 7½ "	27 " 7½ "
Arch, radius of intrados	5 " 0 "	6 " 8 "	8 " 4 "	10 " 0 "	11 " 8 "	13 " 4 "
" " extrados	6 " 0 "	7 " 10 "	9 " 7 "	11 " 4 "	13 " 2 "	14 " 10 "
rise (= one sixth of the span)	1 " 0 "	1 " 4 "	1 " 8 "	2 " 0 "	2 " 4 "	2 " 8 "
thickness at crown	1 " 0 "	1 " 2 "	1 " 3 "	1 " 4 "	1 " 6 "	1 " 6 "
" " the skewback	1 " 7½ "	2 " 3 "	2 " 4 "	2 " 7 "	2 " 10 "	3 " 2 "
" " springing	3 " 3 "	3 " 8 "	4 " 0 "	4 " 3 "	4 " 7 "	5 " 0 "
number of voussoirs	7	7	9	9	13	15
width of voussoirs on intrados	0 11 1/16 "	1 " 2 11/16 "	1 " 2 5/16 "	1 " 5¼ "	1 " 1½ "	1 " 1 11/16 "
Side wall, thickness at springing	3 " 3 "	4 " 2 "	4 " 6 "	4 " 9 "	5 " 1 "	5 " 6 "
" " top of footing	3 " 9 "	5 " 2 "	5 " 6 "	5 " 9 "	6 " 1 "	6 " 6 "
height of, above top of footing	3 " 0 "	4 " 0 "	5 " 0 "	5 " 0 "	5 " 0 "	4 " 8 "
Wing, length for slope of 1½ to 1	none	15 " 0 "	17 " 1 "	17 " 8 "	17 " 4 "	17 " 11 "
thickness at bottom of section E–F	none	3 " 6 "	4 " 0 "	4 " 6 "	4 " 0 "	4 " 9 "
CONTENTS:						
Two end walls, above footing	10 76 cu. yds.	16.58 cu. yds.	19.98 cu. yds.	20.91 cu. yds.	20.73 cu. yds.	23.06 cu. yds.
Arch, per foot of length—inside to inside of end walls	0 589 " "	0.817 " "	1.086 " "	1.365 " "	1.688 " "	1.928 " "
Two side walls, per foot of length—inside to inside of end walls	0 777 " "	1.390 " "	1.858 " "	1.944 " "	2 067 " "	2.080 " "
Four wings, 1	none	19 48 " "	25.07 " "	26.77 " "	25 79 " "	27.57 " "
"	none	5 18 " "	6.27 " "	10.84 " "	8 65 " "	9.06 " "
Paving, per ... t to out of end walls	0.360 cu. yds.	0 346 " "	0 432 " "	0.518 " "	0 605 " "	0.691 " "
Coping, on two end walls	76 8 cu. ft.	78.5 cu. ft.	86.5 cu. ft.	94.5 cu. ft.	102.5 cu. ft.	110 5 cu. ft.

§ 619). The arch is also stable under any position of the moving load, with either a heavy or a light embankment. The joints of the abutment are radial, to prevent any possibility of failure by the sliding of one course on another (see § 674).

Table 53 (page 433) gives the dimensions and contents of various sizes. In each case the rise is one fifth of the span, the central angle is 87° 12′, and the height of the opening is equal to half the span. The paving and coping were each assumed to be 1 foot thick; but for any other thickness it is only necessary to increase or decrease the tabular numbers proportionally. The contents of the wings were computed on the assumption that all the courses were 1 foot thick (see § 557).

650. Quality of Masonry. The masonry of arch culverts is usually divided into two classes; the first consists of the masonry in the wings and end walls (parapet), and the second of the arch stones. The former is classified as first-class or second-class masonry (see § 225). Only the masonry in the arch stones is called arch masonry. The arch stones which show at the end of the arch are called *ring stones,* and the remainder of the arch stones the *arch sheeting.* The arch masonry proper is usually classified as first-class or second-class arch masonry. The distinction between these two classes is usually about as in the specifications below.

651. Specifications.* *Foundations.* "When the bottom of the pit is common earth, gravel, etc., the foundations of arch culverts will generally consist of a pavement formed of stone, not less than twelve inches (12″) in depth, set edgewise, and secured at the ends by deep curbstones which must be protected from undermining by broken stone placed in such quantity and position as the engineer may direct. When the bottom upon which a culvert is to be built is soft and compressible, and where it will at all times be covered with water, timber well hewn, and from eight (8) to twelve inches (12″) in thickness, according to the span of the culvert, shall be laid side by side crosswise upon longitudinal sills; and when the position of the culvert is such that a strong current will be forced through during floods, three courses of sheet piling shall be placed across the foundation—one course at each end, and one in the middle,—which shall be sunk from three (3) to six feet (6′) below the top of the timber, according as the earth is more or less compact."†

652. *First-Class Arch Masonry.* "First-class arch masonry shall be built in accordance with the specifications for first-class masonry [§ 225], with the exception of the arch sheeting and the ring stones. The ring stones shall be

* See also Specifications for Railroad Masonry, Appendix I.

† Pennsylvania Railroad.

TABLE 53.

DIMENSIONS AND CONTENTS OF STANDARD ARCH CULVERTS.

FOR DIAGRAM SEE PLATE VI.

Dimensions not given in the table are the same, for all sizes, as in the diagram.

Items.	Span. 6 ft.	8 ft.	10 ft.	12 ft.	14 ft.	16 ft.
Dimensions:						
End wall, length	11 ft. 2 in.	13 ft. 4 in.	15 ft. 5 in.	17 ft. 6 in.	19 ft. 7 in.	21 ft. 8 in.
height—top of footing to bottom of coping	6 " 0 "	7 " 5 "	9 " 0 "	10 " 6 "	12 " 0 "	13 " 5 "
Coping, length	12 " 2 "	14 " 4 "	16 " 5 "	18 " 6 "	20 " 7 "	22 " 8 "
Arch, radius of intrados	4 " 4.2 "	5 " 9.6 "	7 " 3 "	8 " 8.4 "	10 " 1.8 "	11 " 7.2 "
" " extrados	7 " 2 "	9 " 6 "	12 " 2 "	14 " 8 "	17 " 1 "	20 " 0 "
rise (= one fifth of the span)	1 " 2.4 "	1 " 7.2 "	2 " 0 "	2 " 4.8 "	2 " 9.6 "	3 " 2.4 "
thickness at crown	0 " 10 "	0 " 11 "	1 " 0 "	1 " 1 "	1 " 2 "	1 " 3 "
length of extrados	6 " 7.4 "	8 " 9.9 "	11 " 4.1 "	13 " 2.9 "	15 " 5.4 "	17 " 7.9 "
number of voussoirs	7	9	11	13	13	15
width of voussoirs on intrados	0 " $11\frac{3}{8}$ "	0 " $11\frac{1}{2}$ "	1 " 0 "	1 " $\frac{1}{2}$	1 " $2\frac{1}{4}$ "	1 " $2\frac{3}{8}$ "
Side walls, thickness at springing	2 " 0 "	2 " 6 "	3 " 0 "	3 " 6 "	4 " 0 "	4 " 6 "
" " top of footing	3 " 0 "	3 " 10 "	4 " 8 "	5 " 6 "	6 " 4 "	7 " 2 "
height of, above top of footing	3 " 0 "	4 " 0 "	5 " 0 "	6 " "	7 " 0 "	8 " 0 "
Wing, length for slope of $1\frac{1}{2}$ to 1	11 " 11 "	14 " 6 "	17 " 2 "	19 " 10 "	22 " 5 "	25 " 1 "
thickness at bottom section E-F	2 " 6 "	2 " 6 "	3 " 0 "	4 " 2 "	4 " 10 "	5 " 5 "
Contents:						
Two end walls, above footing	6.671 cu. yds.	9.022 cu. yds.	11.001 cu. yds.	13.828 cu. yds.	16.545 cu. yds.	19.337 cu. yds.
Arch, per foot of length—inside to inside of end walls	0.354 " "	0.504 " "	0.693 " "	1.000 " "	1.297 " "	1.661 " "
Two side walls, per foot of length—inside to inside of end walls	0.555 " "	0.987 " "	1.407 " "	2.000 " "	2.679 " "	3.469 " "
Four wings—slope $1\frac{1}{2}$ to 1	12.99 " "	19.49 " "	30.48 " "	44.08 " "	61.07 " "	81.96 " "
Paving, per foot of length—out to out of end walls	0.185 " "	0.259 " "	0.333 " "	0.407 " "	0.481 " "	0.555 " "
Coping, on two end walls	48.67 cu. ft.	57.33 cu. ft.	65.67 cu. ft.	74.00 cu. ft.	82.33 cu. ft.	90.67 cu. ft.

dressed to such shape as the engineer shall direct. The ring stones and the arch sheeting shall be of stone not less than ten inches (10") thick on the intrados, shall be dressed with three eighths of an inch (⅜") joints, and shall be of the full depth specified for the thickness of the arch; and the joints shall be at right angles to the surface of the intrados. The face of sheeting stones shall be dressed to make a close centering joint. The ring stones and the sheeting shall break joints not less than one foot (1').

"The wings shall be neatly stepped with selected stones of the full width of the wing and of not less than ten inches (10") in thickness, which shall overlap by not less than eighteen inches (18"); or shall be finished with a neatly-capped newel at the free end, and a coping course on the wing. The parapets shall be finished with a coping course not less than ten inches (10") thick and of the full width of the parapet, which shall project six inches (6")."

653. *Second-Class Arch Masonry.* "Second-class arch masonry is the same as second-class masonry [§ 225], with the exception of the arch sheeting. The stones of the arch sheeting shall have a good bearing throughout, and shall be well bonded and of the full depth of the thickness of the arch. No stone shall be less than four inches (4") in thickness on the intrados. Ring stones of all arches over eight feet (8') span shall be dressed according to specifications for first-class arch masonry [§ 651]." *

654. *Paving.* For specifications for Paving, see § 219 (page 148), and also Specifications for Railroad Masonry, Appendix I.

655. **Cost.** §§ 226–38 contain data on the cost of masonry, of which the last is a summary. Table 17 (page 159) contains a detailed statement of the actual cost of the masonry in an arch culvert; and below are the items of the total cost of that culvert.

618 cu. yds. of masonry @ $6.59,	$4,036.85
Excavations—foundations and drainage,	263.36
Sheet piling,	19.69
Concrete,	43.75
Extra allowance on sheeting stones,	20.00
Total cost of culvert,	$4,383.65

The total cost of the culvert per yard of masonry is $7.16,—which is unusually low.

Below is the total actual cost of the 8-ft. culvert (length out to out of end walls = 30 ft.) for which the quantities were estimated in § 645 (page 424).

* Atchison, Topeka and Santa Fé R. R.

Wall masonry—48.7 cu. yds. @ $7.00, ...	$340.90
Arch masonry—28.7 " " " 8.50, ...	243.95
Timber—5,247 ft., B. M., @ $40.00, ...	209.88
Excavating foundations and straightening stream 158 cu. yds. @ 50c., ...	79.00
Total cost of culvert	$873.73

The total cost of this culvert per cubic yard of masonry is $11.29. The average total cost of a number of representative culverts of this style was $11.46 per cubic yard of masonry, being practically constant for all spans.

656. Illinois Central Culverts. Table 54 gives the cost of culverts 25 feet long—out to out of end walls—of various spans of the general plan shown in Plate II, and will be very useful in estimating the cost of such culverts. The quantities of masonry necessary to compute Table 54 were taken from Table 49 (page 425). The prices are believed to be fair averages (see page 160) for the first-class masonry described in § 651. The prices are the same as actually paid by the Illinois Central Railroad, except for arch masonry and excavation, for which $8.50 and 50c. respectively were paid. The prices used in deducing the table are given therein, and hence the results can be modified for prices differing from those there employed by simply taking proportional parts of the tabulated

TABLE 54.

COST OF ILLINOIS CENTRAL ARCH CULVERTS 25 FT. LONG FROM OUT TO OUT OF END WALLS, AND ALSO OF EACH ADDITIONAL FOOT.

FOR DESCRIPTION SEE PAGE 424.

ITEMS.	SPAN.			
	6 ft.	8 ft.	10 ft.	12 ft.
Plain masonry @ $7.00 per cu. yd.	$237.85	$325.29	$424.97	$455.98
Arch masonry @ 8.00 " " "	151.29	191.29	255.29	362.82
Timber and plank at $40 per M. ft., B. M.	150.38	188.12	282.84	268.92
Excavating foundation @ 25c. per cu. yd.	12.48	15.85	19.22	21.38
Total cost of culvert 25 ft. long	$552.00	$720.55	$982.32	$1,109.10
COST OF AN ADDITIONAL FOOT OF LENGTH:				
Plain masonry @ $7.00 per cu. yd.	$3.11	$3.11	$3.11	$3.53
Arch masonry @ 8.00 " " "	6.05	7.65	10.21	14.51
Timber and plank @ $40 per M ft., B. M.	3.36	3.84	5.04	5.88
Excavation @ 25c. per cu. yd.	.32	.40	.48	.56
Total cost of 1 additional foot	$12.84	$15.00	$18.84	$24.48

quantities. The amount of excavation used in computing the table is the mean of the actual quantities for a number of representative culverts as constructed on the above road.

657. Chicago, Kansas and Nebraska Culverts. Table 55 is given to facilitate estimating the cost of culverts of the general form shown in Plate III. The prices are about the average for the respective kinds of work; but in case it is desired to determine the cost for other prices, it is only necessary to increase or decrease the tabular numbers proportionally. The quantities of excavation are, approximately, averages of the actual amounts for a number of similar culverts, and are equivalent to a pit 2 feet 2 inches deep and of an area equal to the area of the foundation. The table includes only one footing course, but in so doing it is not intended to imply that one is always, or even generally, enough. Notice that the culvert in Table 55 is 25⅓ feet long from outside to outside of end walls, and hence is one third of a foot longer than that presented in Table 54.

658. A., T. and S. F. Semi-circular Culverts. Table 56 is similar to the two preceding ones, and shows the cost of the Atchison,

TABLE 55.

COST OF C. K. AND N. ARCH CULVERTS 20 FT. LONG FROM INSIDE TO INSIDE OF COPING, AND ALSO OF EACH ADDITIONAL FOOT OF LENGTH.

FOR DESCRIPTION SEE PAGE 427.

This table includes one footing course.

ITEMS.	SPAN.				
	3 ft.	4 ft.	6 ft.	8 ft.	10 ft.
Plain masonry @ $7.00 per cu. yd.......	$217.98	$397.04	$657.23	$703.43	$853.16
Arch masonry @ 8.00 " " ".......	44.64	79.03	130.78	239.92	346.96
Paving @ 2.00 " " ".......	4.42	6.32	10.16	13.96	17.76
Excavation @ .25 " " ".......	7.44	9.33	12.42	13.95	14.99
Total cost of culvert 20 ft. long ...	$274.48	$491.72	$810.59	$971.26	$1,232.87
COST OF AN ADDITIONAL FOOT OF LENGTH:					
Plain masonry @ $7.00 per cu. yd.......	$4.14	$6.17	$9.33	$9.67	$10.97
Arch masonry @ 8.00 " " ".......	2.30	4.06	6.72	10.06	14.55
Paving @ 2.00 " " ".......	.17	.25	.40	.54	.69
Excavation @ .25 " " ".......	.23	.27	.35	.39	.45
Total cost of 1 additional foot.....	$6.84	$10.75	$16.80	$20.66	$26.66

Topeka and Santa Fé's standard semi-circular arch culvert as given in Plate IV and Table 51 (page 430). The excavation is only approximate, and is computed on the assumption of a pit 2 feet 2 inches deep for the entire foundation including the paved area; *i. e.*, the excavation is computed on the same basis as the two preceding. Notice that this culvert is 23 feet between the outer faces of the end walls, and hence is 1 foot shorter than that of Table 54 and 2½ feet shorter than that of Table 55.

TABLE 56.

COST OF A. T. AND S. F. SEMI-CIRCULAR ARCH CULVERTS 20 FT. LONG FROM INSIDE TO INSIDE OF THE COPING, AND ALSO OF EACH ADDITIONAL FOOT OF LENGTH.

FOR DESCRIPTION SEE PAGE 429.

This table does not include the masonry in the footings.

ITEMS.	SPAN.					
	6 ft.	8 ft.	10 ft.	12 ft.	14 ft.	16 ft.
Plain masonry @ $7.00 per cu. yd.	$325.15	$766.42	$997.14	$1,071.42	$1,328.12	$1,785.91
Arch masonry " 8.00 " " "	140.72	197.28	270.32	356.04	418.40	516.48
Paving " 2.00 " " "	9.88	18.15	16.42	19.68	22.99	26.26
Excavation " .25 " " "	6.93	18.51	16.01	18.21	21.10	24.44
Total cost of culvert 20 ft. long.	$482.68	$990.36	$1,299.89	$1,465.35	$1,790.61	$2,353.09
COST OF AN ADDITIONAL FOOT OF LENGTH:						
Plain masonry @ $7.00 per cu. yd.	$5.44	$8.49	$11.66	$12.44	$14.98	$19.48
Arch masonry " 8.00 " " "	6.12	8.58	11.75	15.48	18.19	22.46
Paving " 2.00 " " "	.52	.69	.86	1.04	1.21	1.38
Excavation " .25 " " "	.28	.32	.40	.44	.48	.54
Total cost of 1 additional foot..	$12.36	$18.08	$24.68	$29.50	$34.86	$43.86

659. A., T. and S. F. Segmental Culverts. Table 57 is similar to the three preceding, and is given to facilitate estimating the cost of segmental arch culverts of the standard form employed by the Atchison, Topeka and Santa Fé Railroad, as shown in Plate V and Table 52 (page 431). The excavation is only approximate, and is computed on the assumption of a pit 2 feet 2 inches deep over the entire foundation, including the paved area. Notice that this culvert is 23 feet between the outer faces of the end walls, and is therefore the same length as that of Table 56.

TABLE 57.

COST OF A. T. AND S. F. SEGMENTAL ARCH CULVERTS 20 FT. LONG FROM INSIDE TO INSIDE OF THE COPING, AND ALSO OF EACH ADDITIONAL FOOT OF LENGTH.

FOR DESCRIPTION SEE PAGE 429.

This table does not include the masonry in the footings.

ITEMS.	SPAN.					
	6 ft.	8 ft.	10 ft.	12 ft.	14 ft.	16 ft.
Plain masonry @ $7.00 per cu. yd......	$183.45	$470.34	$607.99	$657.83	$641.41	$669.13
Arch masonry " 8.00 " " "	99.18	150.83	190.99	229.12	298.64	353.54
Paving " 2.00 " " "	9.88	13.15	16.42	19.68	22.99	25.35
Excavation " .25 " " "	7.33	11.75	14.11	15.17	16.45	17.32
Total cost of culvert 20 feet long. ...	**$299.84**	**$645.57**	**$829.51**	**$921.80**	**$979.49**	**$1,067.05**
COST OF AN ADDITIONAL FOOT OF LENGTH:						
Plain masonry @ $7.00 per cu. yd......	$5.44	$9.73	$12.96	$13.61	$14.47	$14.56
Arch masonry " 8.00 " " "	4.31	6.54	8.30	9.96	12.98	15.38
Paving " 2.00 " " "	.52	.69	.86	1.04	1.21	1.38
Excavation " .25 " " "	.31	.89	.46	.50	.56	.62
Total cost of 1 additional foot........	**$10.58**	**$17.85**	**$22.58**	**$25.11**	**$29.22**	**$31.94**

660. Standard Arch Culvert. Table 58 is given to facilitate the estimation of the cost of culverts of the general form shown in Plate VI. The prices are about the average for the respective kinds of work; but in case it is desired to determine the cost for other prices,

TABLE 58.

COST OF STANDARD ARCH CULVERTS 20 FT. LONG FROM INSIDE TO INSIDE OF THE COPING, AND OF EACH ADDITIONAL FOOT OF LENGTH.

FOR DESCRIPTION SEE PAGE 429.

The masonry in the footings is not included in this table.

ITEMS.	SPAN.					
	6 ft.	8 ft.	10 ft.	12 ft.	14 ft.	16 ft.
Plain masonry @ $7.00 per cu. yd....	$233.11	$330.88	$496.79	$683.55	$912.45	$1,193.35
Arch masonry " 8.00 " " "	65.87	92.72	127.38	184.00	238.64	305.63
Paving " 2.00 " " "	6.83	9.84	12.65	15.47	18.28	20.09
Excavation " .25 " " "	8.24	9.53	12.42	15.42	18.33	20.61
Total cost of culvert 20 feet long ...	**$314.05**	**$442.97**	**$649.19**	**$898.44**	**$1,187.70**	**$1,539.67**
COST OF AN ADDITIONAL FOOT OF LENGTH:						
Plain masonry @ $7.00 per cu. yd....	$3.88	$6.56	$9.85	$14.00	$18.76	$24.28
Arch masonry " 8.00 " " "	2.86	4.08	5.54	8.00	10.38	13.99
Paving " 2.00 " " "	.87	.52	.67	.81	.95	1.11
Excavation " .25 " " "	.21	.26	.31	.36	.41	.46
Total cost of 1 additional foot.......	**$7.82**	**$11.37**	**$16.37**	**$23.17**	**$30.50**	**$39.14**

it is only necessary to increase or decrease the tabular numbers proportionally. The quantities of excavation are, approximately, averages of the actual amounts for a number of similar culverts, and are equivalent to a pit 2 feet 2 inches deep and of an area equal to the area of the foundation. Notice that the culvert in Table 58 is 23 feet between the outer faces of the end walls; and is therefore the same length as that in Tables 56 and 57, and is 1 foot shorter than that of Table 54 and 2½ feet shorter than that of Table 55. Notice also that in Table 58 the height of the opening is in each case half of the span (see Table 53, page 433), while in Tables 56 and 57 the height of the opening is nearly the same for all spans (see Tables 51 and 52, pages 430, 431).

CHAPTER XVIII.

ARCHES.

661. Definitions. Parts of an Arch. *Voussoirs.* The wedge-shaped stones of which the arch is composed; also called the *arch-stones.*

Keystone. The center or highest voussoir or arch-stone.

Soffit. The inner or concave surface of the arch.

Intrados. The concave line of intersection of the soffit, with a vertical plane perpendicular to the axis or length of the arch. See Fig. 110.

Extrados. The convex curve, in the same plane as the intrados, which bounds the outer extremities of the joints between the voussoirs.

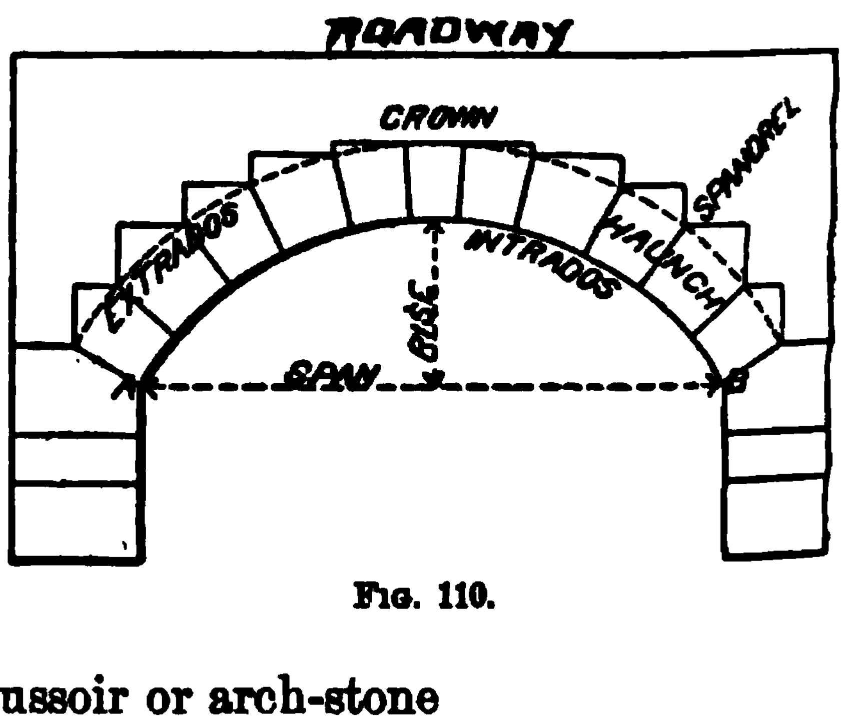

Fig. 110.

Crown. The highest part of the arch.

Skewback. The inclined surface or joint upon which the end of the arch rests.

Abutment. A skewback and the masonry which supports it.

Springing Line. The inner edge of the skewback.

Springer. The lowest voussoir or arch-stone

Haunch. The part of the arch between the crown and the skewback.

Spandrel. The space between the extrados and the roadway. The material deposited in this space is called the *spandrel filling,* and may be either masonry or earth, or a combination of them. In large arches it often consists of several walls running parallel with the roadway, connected at the top by small arches or covered with flat stones, which support the material of the roadway.

Span. The perpendicular distance between the springing lines.

Rise. The vertical distance between the highest part of the intrados and the plane of the springing lines.

Ring Stones. The voussoirs or arch-stones which show at the ends of the arch.

Arch Sheeting. The voussoirs which do not show at the end of the arch.

Backing. Masonry, usually with joints horizontal or nearly so, carried above the skewbacks and outside of the extrados.

String Course. A course of voussoirs extending from one end of the arch to the other.

Coursing Joint. The joint between two adjoining string courses. It is continuous from one end of the arch to the other.

Heading Joint. A joint in a plane at right angles to the axis of the arch. It is not continuous.

Ring Course. The stones between two consecutive series of heading joints.

662. Kinds of Arches. *Circular Arch.* One in which the intrados is a part of a circle.

Semi-circular Arch. One whose intrados is a semi-circle; also called a *full-centered* arch.

Segmental Arch. One whose intrados is less than a semicircle.

Elliptical Arch. One in which the intrados is a part of an ellipse.

Basket-Handle Arch. One in which the intrados resembles a semi-ellipse, but is composed of arcs of circles tangent to each other.

Pointed Arch. One in which the intrados consists of two arcs of equal circles intersecting over the middle of the span. For example, see Figs. 115 and 117, page 447.

Hydrostatic Arch. An arch in equilibrium under the vertical pressure of water.

Geostatic Arch. An arch in equilibrium under the vertical pressure of an earth embankment.

Catenarian Arch. One whose intrados is a catenary.

663. *Right Arch.* A cylindrical arch, either circular or el-

liptical, terminated by two planes, termed *heads* of the arch, at right angles to the axis of the arch. See Fig. 111.

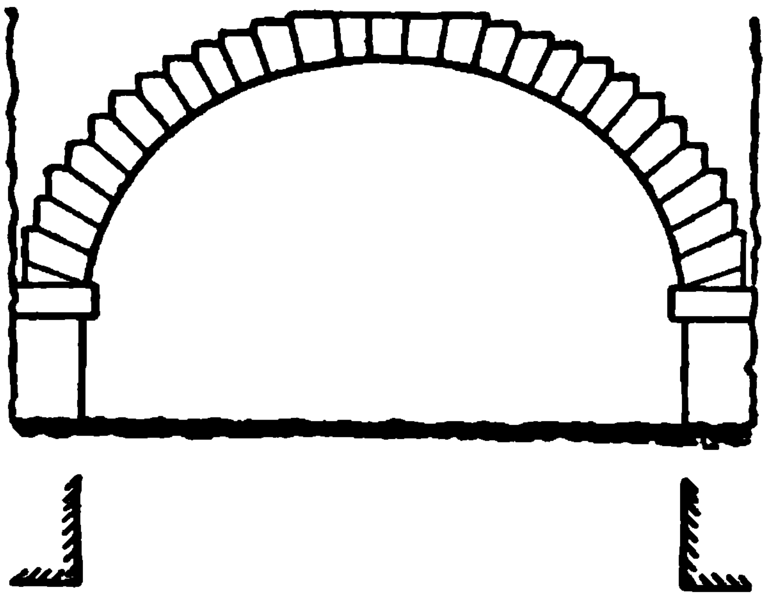

FIG. 111.—RIGHT ARCH.

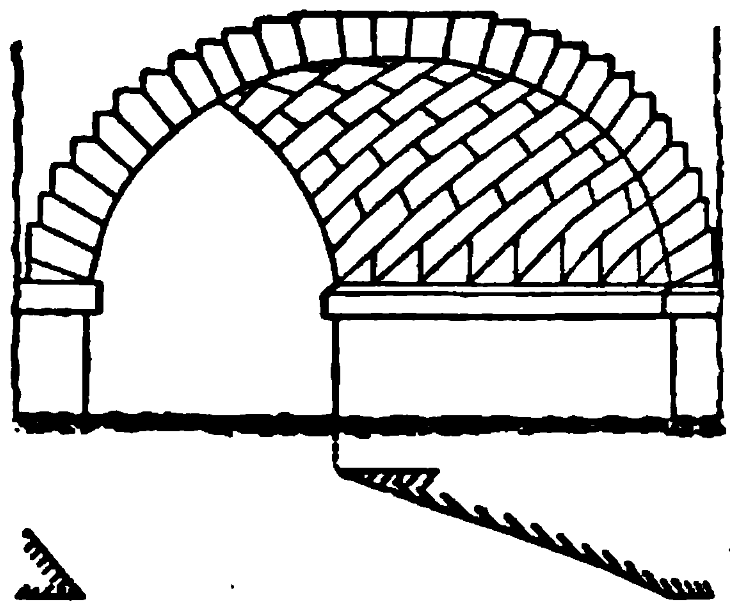

FIG. 112.—SKEW ARCH.

Skew Arch. One whose heads are oblique to the axis. See Fig. 112. Skew arches are quite common in Europe, but are rarely employed in the United States; and in the latter when an oblique arch is required, it is usually made, not after the European method with spiral joints as shown in Fig. 112, but by building a number of short right arches or ribs in contact with each other, each successive rib being placed a little to one side of its neighbor.

Groined and Cloistered Arches. Those formed by the intersection of two or more cylindrical arches. The spans of the intersecting arches may be different, but the rise must be the same in each; and their axes must lie in the same plane, but may intersect at any angle. The *groined arch* is formed by removing those portions of each cylinder which lie under the other and between their common curves of intersection, thus forming a projecting or salient angle on the soffit along these curves. The *cloistered arch* is formed by removing those portions of each cylinder which are above the other and exterior to their common intersection, thus forming re-entrant angles along the same lines.

DOME AND VAULT. If an arch revolves around a vertical through the keystone, a dome is produced; and if it moves in a straight line on the springer, a vault is produced. Hence there are essentially the same kinds of domes and vaults as arches.

Only right arches will be considered in this chapter.

664. Line of Resistance. If the action and reaction between each pair of adjacent arch-stones be replaced by single forces so situated as to be in every way the equivalent of the distributed pressures, the line connecting the points of application of these several forces is the *line of resistance* of the arch. For example, assume that the half arch shown in Fig. 113 is held in equilibrium by the horizontal thrust T—the reaction of the right-hand half of the arch—applied at the middle of the joint CF. Assume also that the

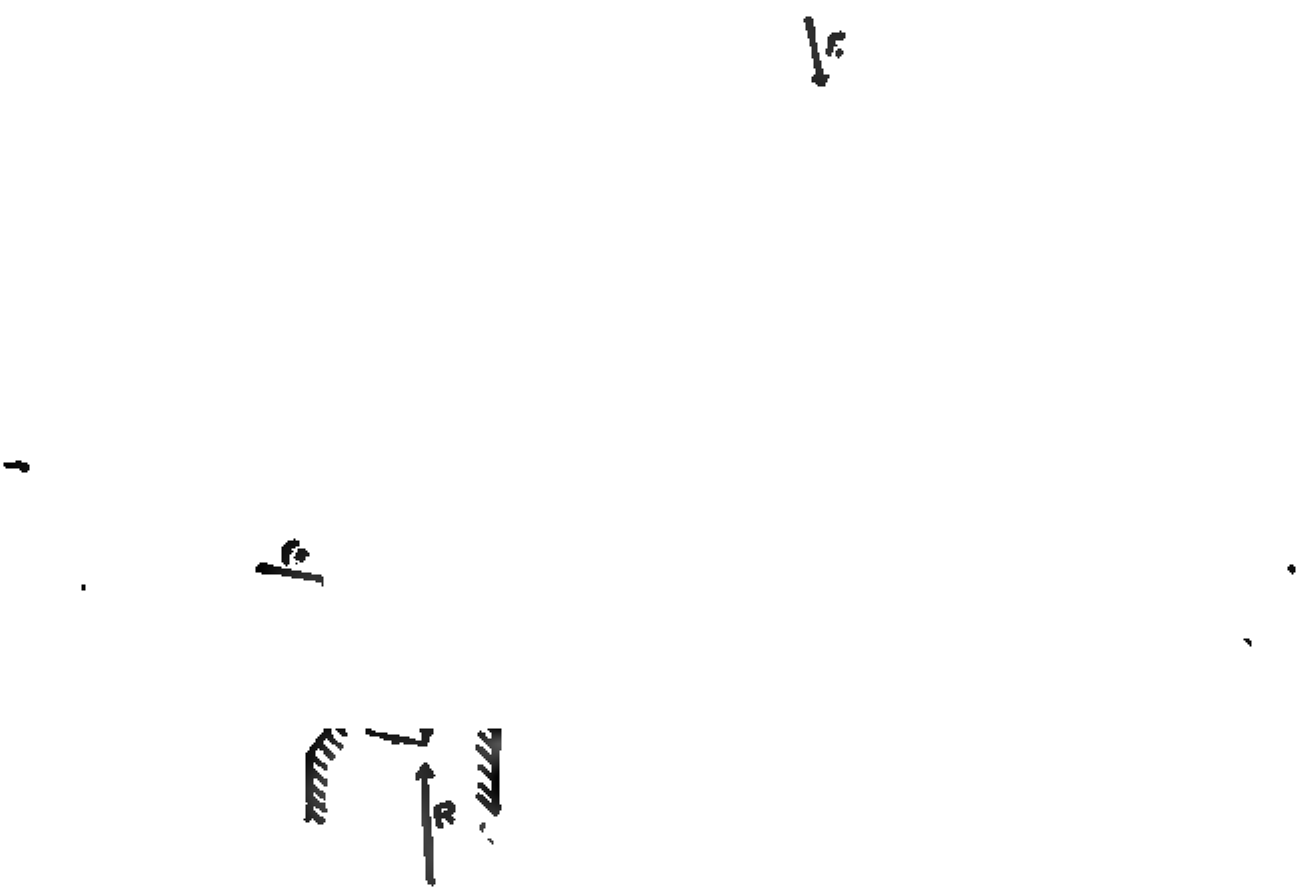

FIG. 113.

several arch-stones fit mathematically, and that there is no adhesion of the mortar. The forces F_1, F_2, F_3, and F_4 represent the resultants of all the forces (including the weight of the stone itself) acting upon the several voussoirs. The arch-stone $CIHF$ is in equilibrium under the action of the three forces, T, F_1, and the reaction of the voussoir $IHEG$. Hence these three forces must intersect in a point, and the direction of R_1—the resultant pressure between the voussoirs $CIHF$ and $IHEG$—can be found graphically as shown in Fig. 113. The point of application of R_1 is at b—the point where R_1 intersects the joint HI. The voussoir $GEHI$

is in equilibrium under the action of R_1, F_2, and R_2—the resultant reaction between *GEHI* and *GEDH*,—and hence the direction, the amount, and the point of application (c) of R_2 can be determined as shown in the figure. R_3 and R_4 are determined in the same manner as R_1 and R_2.

The points a, b, c, d, and e, called *centers of pressure*, are the points of application of the resultants of the pressure on the several joints; or they may be regarded as the *centers of resistance* for the several joints. In the latter case the line *abcde* would be called the *line of resistance*, and in the former the *line of pressure*. Strictly speaking, the line of resistance is a continuous curve circumscribing the polygon *abcde*. The greater the number of joints the nearer the polygon *abcde* approaches this curve. Occasionally the polygon *mnop* is called the line of resistance. The greater the number of joints the nearer this line approaches the line of resistance as defined above. For an infinite number of joints the polygons *abcde* and *mnop* coincide with the curved line of resistance, a, b, c, d, and e being common to all three.

Notice that if the four geometrical lines *ab*, *bc*, *cd*, and *de* were placed in the relative position shown in Fig. 113, and were acted upon by the forces T, F_1, F_2, F_3, F_4, and R, as shown, they would be in equilibrium; and hence the line *abcde*, or rather a curve passing through the points a, b, c, d, and e, is sometimes called a *linear arch*.

ART. 1. THEORY OF THE ARCH.

665. The theory of the masonry arch is one of great complexity. Numerous volumes have been written on this subject, and it still occupies the attention of mathematicians. No attempt will be made here to give an exhaustive treatise on the arch; but the fundamental principles will be stated as clearly as possible, and the principal solutions of the problem which have been proposed from time to time will be explained and their underlying assumptions pointed out.

666. THE EXTERNAL FORCES. It is clear that before we can find the strains in a proposed arch and determine its dimensions, we must know the load to be supported by it. In other words, the strength and stability of a masonry arch depend upon the

position of the line of resistance; and before this can be determined, it is necessary that the external forces acting upon the arch shall be fully known, *i. e.*, that (1) the point of application, (2) the direction, and (3) the intensity of the forces acting upon each voussoir shall be known. Unfortunately, the accurate determination of the outer forces is, in general, an impossibility.

1. If the arch supports a fluid, the pressure upon the several voussoirs is perpendicular to the extrados, and can easily be found; and combining this with the weight of each voussoir gives the several external forces. This case seldom occurs in practice.

2. If the arch is surmounted by a masonry wall, as is frequently the case, it is impossible to determine, with any degree of accuracy, the effect of the spandrel walls upon the stability of the arch. It is usually assumed that the entire weight of the masonry above the soffit presses vertically upon the arch; but it is known certainly that this is not the case, for with even dry masonry a part of the wall will be self-supporting. The load supported by the arch can be computed roughly by the principle of § 250 (p. 168); but, as this gives no idea of the manner in which this pressure is distributed, it is of but little help. The error in the assumption that the entire weight of the masonry above the arch presses upon it is certainly on the safe side; but if the data are so rudely approximate, it is useless to attempt to compute the strains by mathematical processes. The inability to determine this pressure constitutes one of the limitations of the theory of the arch.

Usually it is virtually assumed that the extradosal end of each voussoir terminates in a horizontal and vertical surface (the latter may be zero); and therefore, since the masonry is assumed to press only vertically, there are no horizontal forces to be considered. But as the extrados is sometimes a regular curve, there would be active horizontal components of the vertical pressure on this surface; and this would be true even though the spandrel masonry were divided by vertical joints extending from the extrados to the upper limit of the masonry. Further, even though no active horizontal forces are developed, the passive resistance of the spandrel masonry—either spandrel walls or spandrel backing—materially affects the stability of an arch. Experience shows that most arches sink at the crown and rise at the haunches when the centers are removed (see Fig. 116, p. 447), and hence the resistance of the spandrel masonry will

materially assist in preventing the most common form of failure. The efficiency of this resistance will depend upon the execution of the spandrel masonry, and will increase as the deformation of the arch ring increases. It is impossible to compute, even roughly, the horizontal forces due to the spandrel masonry.

Further, in computing the strains in the arch, it is usually assumed that the arch ring alone supports the masonry above it; while, as a matter of fact, the entire masonry from the intrados to the top of the wall acts somewhat as an arch in supporting its own weight.

3. If the arch supports a mass of earth, we can know neither the amount nor the direction of the earth pressure with any degree of accuracy (see Chap. XIV—Retaining Walls,—particularly § 527, page 339). We do know, however, that the arch does not support the entire mass above it (see §§ 618–20). No one ever thinks of trying to make a tunnel arch strong enough to sustain the weight of the entire mass above it.

In the theory of the masonry arch, the pressure of the earth is usually assumed to be wholly vertical. That the pressure of earth gives, in general, active horizontal forces appears to be unquestionable. An examination of Fig. 113 (page 443) will show how the horizontal forces add stability to an arch ring whose rise is equal to or less than half the span. It is clear that for a certain position and intensity of thrust T, the line of resistance will approach the extrados nearer when the external forces are vertical than when they are inclined. We know certainly that the passive resistance of the earth adds materially to the stability of masonry arches; for the arch rings of many sewers which stand without any evidence of weakness are in a state of unstable equilibrium, if the vertical pressure of the earth immediately above it be considered as the *only* external force acting upon it.

667. Method of Failure of Arches. A masonry arch may yield in any one of three ways, viz.: (1) by the crushing of the stone, or (2) by the sliding of one voussoir on another, or (3) by rotation about an edge of some joint. 1. An arch will fail if the pressure on any part is greater than the crushing strength of the material composing it. 2. Figs. 114 and 115 represent the second method of failure; in the former the haunches of the arch *slide*

out and the crown slips down, and in the latter the reverse is shown. If the rise is less than the span and the arch fails by the sliding of one voussoir on the other, the crown will usually sink; but if the rise is more than the span, the haunches will generally

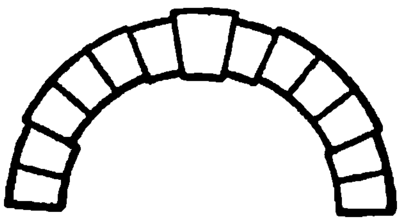

FIG. 114.

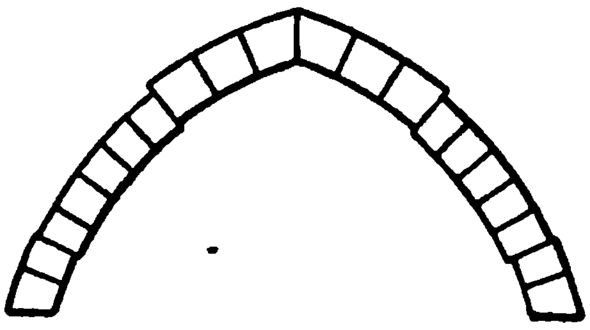

FIG. 115.

be pressed inward and the crown will rise. 3. Figs. 116 and 117 show the two methods by which an arch may give way by rotation

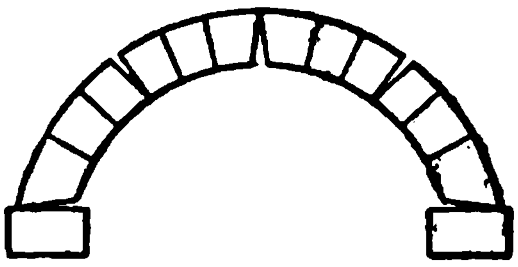

FIG. 116.

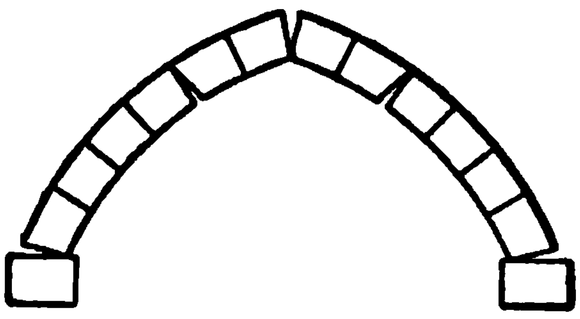

FIG. 117.

about the joints. As a rule the first case is most frequent for flat arches and the second for pointed ones.

However, more arches fail on account of unequal settlement of the foundation than because of a faulty design of the arch proper.

668. CRITERIA OF SAFETY. There are three criteria, corresponding to the three modes of failure, by which the stability of an arch may be judged. (1) To prevent overturning, it is necessary that the line of resistance shall everywhere lie between the intrados and the extrados. (2) To prevent crushing, the line of resistance should intersect each joint far enough from the edge so that the maximum pressure will be less than the crushing strength of the masonry. (3) To prevent sliding, the angle between the line of resistance and the normal to any joint should be less than the angle of repose ("angle of friction") for those surfaces; that is to say, the tangent of the angle between the line of resistance and the normal to any joint should be less than the co-efficient of friction (§ 489).

669. Stability against Rotation. An arch composed of incompressible voussoirs can not fail by rotation as shown in Fig. 116, unless the line of resistance touches the intrados at two points and the extrados at one higher intermediate point (see Fig. 120, page 454); and an arch can not fail by rotation as shown in Fig. 117, unless the line of resistance touches the extrados at two points and the intrados at one higher intermediate point (see Fig. 120). The factor of safety against rotation about any point is equal to half the length of the joint divided by the distance between the center of pressure and the center of the joint; that is to say,

$$\textit{the factor of safety} = \frac{\frac{1}{2}\,l}{d}, \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which l is the length of the joint and d the distance between the center of pressure and the center of the joint. For example, if the center of pressure is at one extremity of the middle third of the joint, $d = \frac{1}{6}\,l$; and, by equation (1), the factor of safety is three. If the center of pressure is $\frac{1}{4}\,l$ from the middle of the joint, the factor of safety is two.

It is customary to require that the line of resistance shall lie within the middle third of the arch ring, which is equivalent to specifying that the minimum factor of safety for rotation shall not be less than three.

670. Stability against Crushing. The method of determining the pressure on any part of a joint has already been discussed in the chapter on masonry dams (see pp. 320–26). When the total pressure and its center are known, the maximum pressure at any part of the joint is given by formula (23), page 323. It is

$$P = \frac{W}{l} + \frac{6\,W d}{l^2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which P is the maximum pressure on the joint per unit of area; W is the total normal pressure on the joint per unit of length of the arch; l is the depth of the joint, *i. e.*, the distance from intrados to extrados; and d is the distance from the center of pressure to the middle of the joint. This formula is general, provided the

masonry is capable of resisting tension; and if the masonry is assumed to be incapable of resisting tension, it is still general, provided d does not exceed $\frac{1}{6}\, l$.

For the case in which the masonry is incapable of resisting tension and d exceeds $\frac{1}{6}\, l$, the maximum pressure is given by formula (24), page 324. It is

$$P = \frac{2\, W}{3\,(\frac{1}{2}\, l - d)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If the line of resistance for any arch can be drawn, the maximum pressure can be found by (1) resolving the resultant reaction perpendicular to the given joint, and (2) measuring the distance d from a diagram of the arch similar to Fig. 113 (page 443), and (3) substituting these data in the proper one of the above formulas (the one to be employed depends upon the value of d), and computing P.* This pressure should not exceed the compressive strength of the masonry.

It is customary to prescribe that the line of resistance shall lie within the middle third of each joint, and also that the result obtained by dividing the total pressure by the area of the joint shall not be *more than* one twentieth of the ultimate crushing strength of the stone. Under these conditions the maximum pressure is twice the mean, and hence using the above limits is equivalent to saying that the maximum pressure shall not be more than one tenth of the ultimate crushing strength of the stone. The mean pressure in arches is usually not more than one fortieth or one fiftieth, and sometimes only one hundredth, of the ultimate compressive strength of the stone or brick of which it is constructed.

671. *Unit Pressure.* In the present state of our knowledge it is not possible to determine the value of a safe and not extravagant unit working-pressure. The customary unit appears less extravagant, when it is remembered (1) that the crushing strength of masonry is considerably less than that of the stone or brick of which it is composed (see §§ 221–22 and §§ 246–47 respectively), and that we have no definite knowledge concerning either the ultimate or the safe crushing strength of stone masonry (§ 223) and but little

* For a numerical example of the method of doing this, see 2, § 690.

concerning that of brickwork (§ 248); and (2) that all the data we have on crushing strength are for a load perpendicular to the pressed surface, while we have no experimental knowledge of the effect of the component of the pressure parallel to the surface of the joint, although it is probable that this component would have somewhat the same effect upon the strength of the voussoirs as a sheet of lead has when placed next to a block of stone subjected to compression (§ 9).

On the other hand, there are some considerations which still further increase the degree of safety of the usual working-pressure. (1) When the ultimate crushing strength of stone is referred to, the crushing strength of cubes is intended, although the blocks of stone employed in actual masonry have less thickness than width, and hence are much stronger than cubes (see § 14, paragraph 2 § 60, and § 273). To prevent the arch stones from flaking off at the edges, the mortar is sometimes dug out of the outer edge of the joint. This procedure diminishes the area under pressure, and hence increases the unit pressure; but, on the other hand, the edge of the stone which is not under pressure gives lateral support to the interior portions, and hence increases the resistance of that portion (see § 273). It is impossible to compute the relative effect of these elements, and hence we can not theoretically determine the efficiency of thus relieving the extreme edges of the joint. (2) The preceding formulas (2 and 3) for the maximum pressure neglect the effect of the elasticity of the stone; and hence the actual pressure must be less, by some unknown amount, than that given by either of the formulas.

672. Notice that the distance which the center of pressure may vary from the center of the joint without the masonry's being crushed depends upon the ratio between the ultimate crushing strength and the mean pressure on the joint. In other words, if the mean pressure is very nearly equal to the ultimate crushing strength, then a slight departure of the center of pressure from the center of the joint may crush the voussoir; but, on the other hand, if the mean pressure is small, the center of pressure may depart considerably from the center of the joint without the stone's being crushed. This can be shown by equation (2), page 448. If both P and $\frac{W}{l}$ are large, d must be small; but if P is large and

$\frac{W}{l}$ small, then d may be large. Essentially the same result can be deduced from equation (3), page 449.

Even though the line of resistance approaches so near the edge of the joint that the stone is crushed, the stability of the arch is not necessarily endangered. For example, conceive a block of stone resting upon an incompressible plane, AB, Fig. 118, and assume that the center of pressure is at N. Then the pressure is applied over an area projected in $A\,V$, such that $AN = \frac{1}{3}\,A\,V$. The pressure at A is represented by AK, and the area of the triangle AKV represents the total pressure on the joint. Assume that AK is the ultimate crushing strength of the stone, and that the center of pressure is moved to N'. As before, the pressure is borne on an area projected in $A\,V'$, such that $AN' = \frac{1}{3}\,A\,V'$. The pressure in the vicinity of A is uniform and equal to the crushing strength AK; hence the total pressure on the joint is represented by the area of the figure $AKG\,V$, which has its center of gravity in the vertical through N'. Eventually, when the center of pressure approaches so near A that the area in which the stone is crushed becomes too great, the whole block will give way and the arch will fall.*

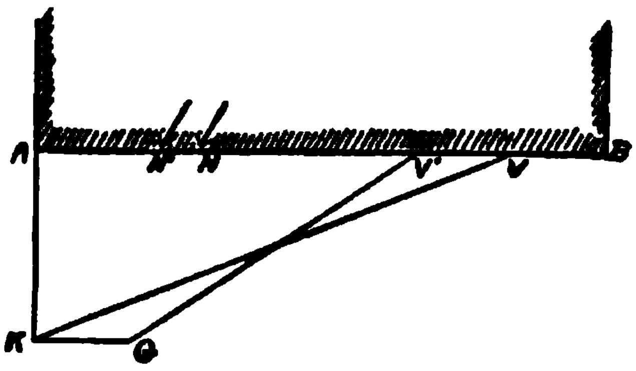

FIG. 118.

673. *Open Joints.* It is frequently prescribed that the line of resistance shall pass through the middle third of each joint, "so that the joint may not open on the side most remote from the line of resistance." If the line of resistance departs from the middle third, the remote edge of the joint will be in tension; but since cement mortar is now quite generally employed, if the masonry is laid with ordinary care, the joint will be able to bear considerable tension (see Table 13, page 94); and hence it does not necessarily follow that the joint will open.

* Rankine says: "It is true that arches have stood, and still stand, in which the centers of resistance of joints fall beyond the middle third of the depth of the arch ring; but the stability of such arches is either now precarious, or must have been precarious while the mortar was fresh." The above is one reason why the stability of the arch is not necessarily precarious, and other reasons are found in § 666 and also in the subsequent discussion. A reasonable theory of the arch will not make a structure appear instable which shows every evidence of security.

If the line of pressure departs from the middle third and the mortar is incapable of resisting tension, the joint will open on the side farthest from the line of resistance. For example, if the center of pressure is at N, Fig. 118, then a portion of the joint $A\,V$ $(= 3\,A\,N)$ is in compression, while the portion VB has no force acting upon it; and hence the yielding of the portion $A\,V$ will cause the joint to open a little at B. This opening will increase as the center of pressure approaches A, and when the material at that point begins to crush the increase will become comparatively rapid.

Notice that the opening of a joint does not indicate that the stability of the arch is in danger. In most cases, an open joint is no serious matter, particularly if it is in the soffit. If in the extrados, it is a little more serious, since water might get into it and freeze. To guard against this danger, it is customary to cover the extrados with a layer of puddle or some coating impervious to water (§ 264). Notice also that if there are no open joints in an arch, it is probable that the actual line of resistance lies within the middle third of the arch ring.

674. Stability against Sliding. If the effect of the mortar is neglected, an arch is stable against sliding when the line of resistance makes with the normal an angle less than the angle of friction. According to Table 36 (page 315) the co-efficient of friction of masonry under conditions the most unfavorable for stability—*i. e.*, while the mortar is wet—is about 0.50, which corresponds to an angle of friction of about 25°. Hence if the line of pressure makes an angle with the normal of more than 25°, there is a possibility of one voussoir's sliding on the other. This possibility can be eliminated by changing the joints to a direction more nearly at right angles to the line of pressure.

However, there is no probability that an arch will receive its full load before the mortar has begun to set; and hence the angle of friction is virtually much greater than 25°. It is customary to arrange the joints of the arch at least nearly perpendicular to the line of resistance, in which case little or no reliance is placed on the resistance of friction or the adhesion of the mortar.

675. Conclusion. From the preceding discussion, it will be noticed that the factors of stability for rotation and for crushing are dependent upon each other; while the factor for sliding is independent of the other conditions of failure, and is dependent

only upon the direction given to the joints. A theoretically perfect design for an arch would be one in which the three factors of safety were equal to each other and uniform throughout the arch. As arches are ordinarily built, the factor for rotation is about three, or a little more; the nominal factor for crushing is ten to forty; and the nominal factor for sliding is one and a half to two.

It is evident that before any conclusions can be drawn concerning the strength or stability of a masonry arch, the position of the line of resistance must be known; or, at least, limits must be found within which the true line of resistance must be proved to lie.

676. Location of the True Line of Resistance. The determination of the line of resistance of a semi-arch requires that the external forces shall be fully known, and also that (1) the amount, (2) the point of application, and (3) the direction of the thrust at the crown shall be known. The determination of the external forces is a problem independent of the theory of the arch; and for the present it will be assumed that they are fully known, although as a matter of fact they can not be known with any considerable degree of accuracy (see § 666).

Each value for the intensity of the thrust at the crown gives a different line of resistance. For example, in Fig. 113 (page 443), if the thrust T be increased, the point b—where R_1 intersects the plane of the joint HI—will approach I; and consequently c, d, and e will approach G, H, and A respectively. If T be increased sufficiently, the line of pressure will pass through A or H (usually the former, this depending, however, upon the dimensions of the arch and the values and directions of F_1, F_2, and F_3), and the arch will be on the point of rotating about the outer edge of one of these joints. This value of T is then the maximum thrust at a consistent with stability of rotation about the outer edge of a joint, and the corresponding line of resistance is the line of resistance for maximum thrust at a. Similarly, if the thrust T be gradually decreased, the line of resistance will approach and finally intersect the intrados, in which case the thrust is the least possible consistent with stability of rotation about some point in the intrados. The lines of resistance for maximum and minimum thrust at a are shown in Fig. 119 (page 454).

If the point of application of the force T be gradually lowered and at the same time its intensity be increased, a line of resistance

may be obtained which will have one point in common with the intrados. This is the line of resistance for maximum thrust at the crown joint. Similarly, if the point of application of T be gradually raised, and at the same time its intensity be decreased, a line of resistance may be obtained which will have one point in common with the extrados. This is the line of resistance for minimum thrust at the crown joint. The lines of resistance for maximum and minimum thrust at the crown are shown in Fig. 120.

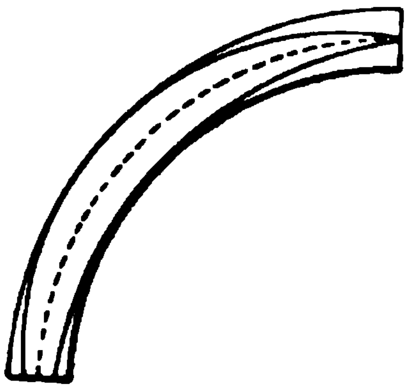

FIG. 119.

Similarly each direction of the thrust T will give a new line of resistance. In short, every different value of each of the several factors, and also every combination of these values, will give a different position for the line of resistance. Hence, the problem is to determine which of the infinite number of possible lines of resistance is the actual one. This problem is indeterminate, since there are more unknown quantities than conditions (equations) by which to determine them. To meet these difficulties and make a solution of the problem possible, various hypotheses have been made ; but there is no unanimity of opinion among authorities regarding the position of the true line of resistance. Some of these hypotheses will now be considered briefly.

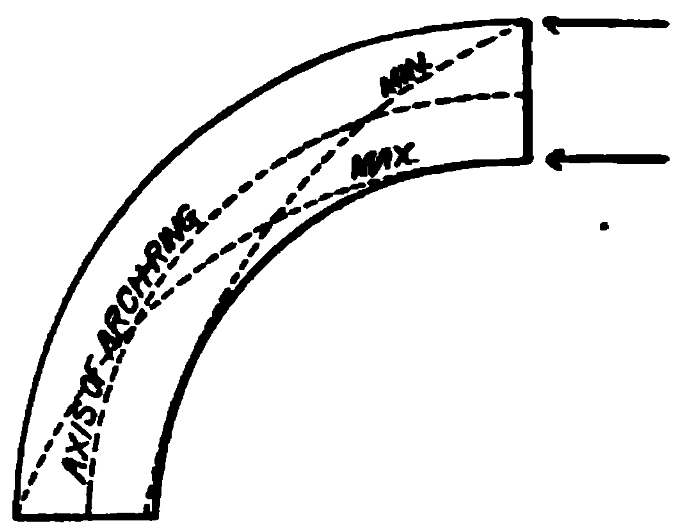

FIG. 120.

677. Hypothesis of Least Pressure. Some writers have assumed the true line of resistance to be that which gives the smallest absolute pressure on any joint. This principle is a meta-physical one, and leads to results unquestionably incorrect. Of the four hypotheses here discussed this is the least satisfactory, and the least frequently employed. It will not be considered further.

For an explanation of Claye's method of drawing the line of pressure according to this theory, see Van Nostrand's Engineering Magazine, vol. xv, pp. 33–36. For a general discussion of the theory of the arch founded on this hypothesis, see an article by Pro-

fessor Du Bois in Van Nostrand's Engineering Magazine, vol. xiii, pp. 341–46, and also Du Bois's "Graphical Statics," Chapter XV.

678. Hypothesis of Least Thrust at the Crown. According to this hypothesis the true line of resistance is that for which the thrust at the crown is the least possible consistent with equilibrium. This assumes that the thrust at the crown is a passive force called into action by the external forces; and that, since there is no need for a further increase after it has caused stability, it will be the least possible consistent with equilibrium.

This principle alone does not limit the position of the line of resistance; but, if the external forces are known and the direction of the thrust is *assumed*, this hypothesis furnishes a condition by which the line of resistance corresponding to a minimum thrust can be found by a tentative process. The principle of least crown thrust was first proposed by Moseley,* was amplified by Scheffler,† and has been adopted more generally by writers and engineers than any other.

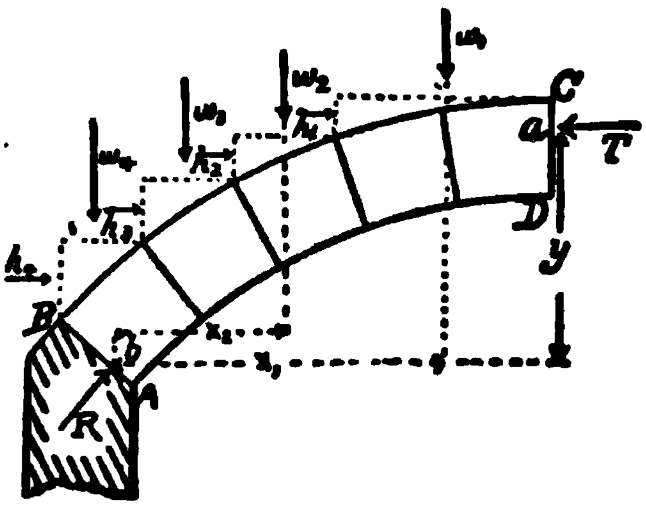

FIG. 121.

679. The half arch shown in Fig. 121 is held in equilibrium by (1) the vertical forces, w_1, w_2, etc., (2) by the horizontal forces h_1, h_2, etc., (3) by the reaction R at any joint, and (4) by the thrust T at the crown. The direction of R is immaterial in this discussion. Let a and b represent the points of application of T and R, respectively, although the location of these points is yet undetermined. Let

T = the thrust at the crown;

x_1 = the horizontal distance from b to the line of action of w_1;

x_2 = the same for w_2, etc.;

* Philosophical Magazine, Oct., 1833—see Moseley's Mechanical Principles of Engineering, 2d American ed., p. 430.

† "Theorie der Gewölbe, Futtermauern, und eisernen Brücken," Braunschweig, 1857. A French translation of this work is entitled "Traité de la Stabilité des constructions; 1re partie, Théorie des Voutes et des Murs de Soutenement," Paris, 1864. Cain's "A Practical Theory of Voussoir Arches"—No. 12 of Van Nostrand's Science Series—New York, 1874, is an exposition of a theory of the arch based upon this hypothesis.

y = the perpendicular distance from b to the line of action of T;
k_1 = the perpendicular distance from b to the line of action of h_1; k_2 = the same for h_2; etc.

Then, by taking moments about b, we have

$$Ty = w_1 x_1 + w_2 x_2 + \text{etc.} + h_1 k_1 + h_2 k_2 + \text{etc.}; \quad . \quad (4)$$

hence

$$T = \frac{\Sigma w x}{y} + \frac{\Sigma h k}{y} \ldots \ldots \ldots \ldots \quad (5)$$

1. The value of T depends upon $\Sigma h k$—the sum of the moments of the horizontal component of the external forces;—but we know neither the nature of the material over the arch nor the value of $\Sigma h k$ for any particular material (see §§ 527–31). In discussing and applying this principle, the term $\Sigma h k$ is usually neglected. Ordinarily this gives an increased degree of stability; but this is not necessarily the case. The omission of the effect of the horizontal component makes the computed value of T less than it really is, and causes the line of resistance found on this assumption to approach the *intrados* at the haunches nearer than it does in fact; and hence the conditions may be such that the actual line of resistance will be unduly near the *extrados* at the haunches, and consequently endanger the arch in a new direction.

2. For simplicity of discussion, and because the error involved in the discussion immediately to follow is immaterial, we will temporarily omit the effect of the horizontal components of the external forces. If the horizontal forces are disregarded, equation (5) becomes

$$T = \frac{\Sigma w x}{y} \ldots \ldots \ldots \quad (6)$$

From equation (6) we see that, other things remaining the same, the larger y the smaller T; and hence, for a minimum value of T, a should be as near c as is possible without crushing the stone (see §§ 670–72). Usually it is assumed that ac is equal to one third of the thickness of the arch at the crown; and hence the average pressure per unit of area is to be equal to one half of the assumed unit working pressure; or, in other words, twice the thrust T divided by the thickness of the crown is to be equal to the unit working pressure.

3. To determine y, it is necessary that the direction of T should be known. It is usually assumed that T is horizontal. If the arch is symmetrical and is loaded uniformly over the entire span, this assumption is reasonable ; but if the arch is subject to heavy moving loads, as most are, the thrust at the crown is certainly not horizontal, and can not be determined.

4. If the joint $A\ B$ is horizontal, then b is to be taken as near A as is consistent with the crushing strength of the stone, or at, say, one third of the length of the joint $A\ B$ from A. Notice that if the springing line is inclined, as in general it will be (see last two paragraphs of § 682, p. 463), moving b toward A decreases x, and will at the same time increase y. Hence the position of b corresponding to a minimum value of T can be found only by trial. It is usual, however, to assume that Ab is one third of AB, whatever the inclination of the joint.

680. *Joint of Rupture.* The joint of rupture is that joint for which the tendency to open at the extrados is the greatest. The joint of rupture of an arch is analogous to the dangerous section of a beam. Practically, the joint of rupture is the springing line of the arch, the arch masonry below that joint being virtually only a part of the abutment.

That no joint may open at the extrados, the thrust at the crown must be at least equal to the maximum value of T as determined by equation (5), page 456. If the thrust is less than this, the joint of rupture will open at the extrados; and a greater value is inconsistent with the hypothesis of minimum crown thrust. Since the moment of the horizontal components of the external forces is indeterminable, the position of the true joint of rupture can be found only by trial for assumed values and positions of the horizontal forces.

681. As an example, assume that it is required to determine the joint of rupture of the 16-foot arch shown in Fig. 122, which is the standard form employed on the Chicago, Kansas & Nebraska R. R. (see page 427 and Plate III). Assume that the arch supports an embankment of earth extending 10 feet above the crown, and that the earth weighs 100 pounds per cubic foot and the masonry 160. For simplicity, consider a section of the arch only a foot thick perpendicular to the plane of the paper. The half-arch ring and the earth embankment above it are divided into eight sections,

which for a more accurate determination of the joint of rupture are made smaller near the supposed position of that joint. The weight of the first section rests upon the first joint, that of the first two upon the second joint, etc. The values and the positions of

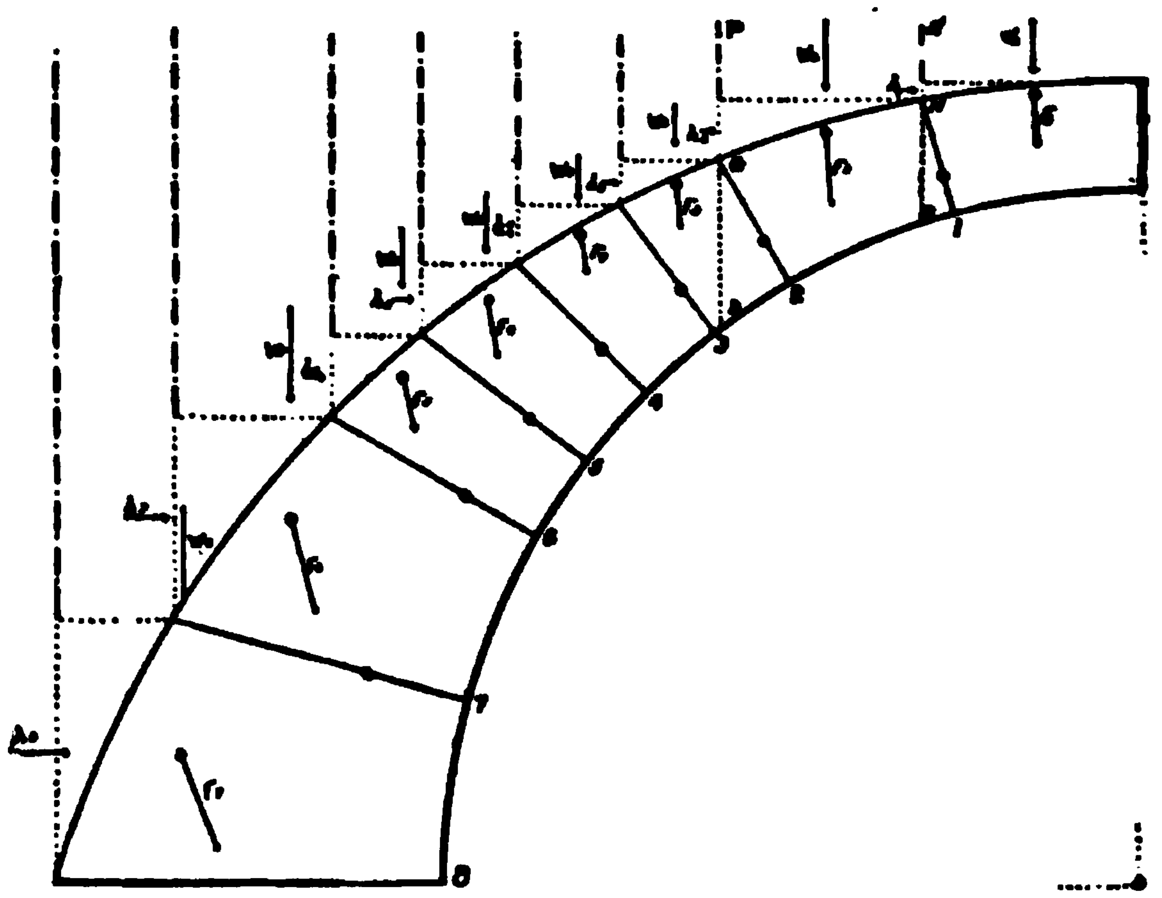

FIG. 122.

the lines of action of the weights of the several sections are given in the second and third columns of Table 59.*

* The center of gravity of the arch stone is found by the method explained in § 494 (page 318); and the center of gravity of the prism of earth resting upon each arch stone may, without sensible error, be taken as acting through its medial vertical line. The center of gravity of the combined weight of the arch stone and the earth resting upon it may be found by either of the two following methods, of which the first is the shorter and more accurate:

1. The center of gravity of the two masses may be found by the following well-known principle of analytical mechanics:

$$\bar{x} = \frac{w_1 x_1 + w_2 x_2}{w_1 + w_2}, \quad \ldots \ldots \ldots \ldots \quad (7)$$

in which $\bar{x}$ is the horizontal distance from any point, say the crown, to the vertical through the center of gravity of the combined masses, w_1 and w_2 are the weights of the two masses, and x_1 and x_2 the horizontal distances from any point, say the crown, to the verticals through the centers of gravity of the separate masses respectively. The same method can be employed for finding the center of gravity of any number of masses, by simply adding the corresponding term or terms in the numerator and the denominator of equation (7).

2. Since the principles employed in the second method of finding the center of gravity of each arch stone and its load are frequently employed, in one form or

TABLE 60.

TO FIND THE JOINT OF RUPTURE OF THE ARCH RING SHOWN IN FIG. 123.

No. of the joint, counting from the one next to the crown.	Data for Vertical Forces.		Data for Horizontal Forces.		Position of the Center of Pressure for each Joint.		Thrust at the Crown.		
	Intensity of the force.	Horizontal distance of point of application from the crown joint.	Intensity of the force.	Vertical distance of point of application from the top of the crown joint.	Horizontal distance from the crown joint.	Vertical distance from the top of the crown joint.	$\frac{\Sigma w x}{y}$	$\frac{\Sigma h b}{y}$	Total thrust.
	Lbs.	Feet.	Lbs.	Feet.	Feet.	Feet.	Lbs.	Lbs.	Lbs.
1	2,968	1.20	66	0.10	2.20	1.16	8,866	94	8,960
2	3,045	3.57	243	0.55	4.27	1.86	7,744	308	8,052
3	1,644	5.33	192	1.17	5.27	2.48	8,518	424	8,942
4	1,716	6.45	259	1.78	6.17	3.11	8,748	662	9,400
5	1,895	7.50	315	2.53	6.96	3.90	8,577	700	9,277
6	1,696	8.47	415	3.40	7.71	4.81	8,407	941	9,348
7	3,969	9.77	1,080	5.02	8.65	6.84	7,506	1,407	8,911
8	4,096	11.05	1,694	7.70	9.50	9.35	5,990	1,968	7,973

another, in discussions of the stability of the masonry arch, this method will be explained a little more fully than is required for the problem in hand.

The first step is to reduce the actual load upon an arch (including the weight of the arch ring itself) to an equivalent homogeneous load of the same density as the arch ring. The upper limit of this imaginary loading is called the *reduced-load contour*. For example, suppose it is required to find the reduced-load contour for the arch loaded as in Fig. 123. Assume that the weight of the arch ring is 160 pounds per

FIG. 123.

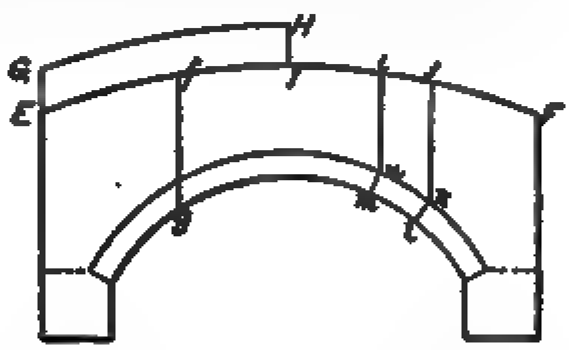

FIG. 124.

cubic foot; that of the rubble backing, 140; and that of the earth, 100. Then the ordinate at a to the load contour of an equivalent load of the density of the arch ring is equal to $a\,b + b\,c\,\frac{140}{160} + c\,d\,\frac{100}{160} =$, say, $g\,f$. The value of $g\,f$ is laid off in Fig. 124. Computing the ordinates for other points in the load contour gives the line $E\,F$, Fig. 124, which is the reduced-load contour for the load shown in Fig. 123. The area between the intrados and the reduced-load contour is proportional to the load on the arch. In a similar manner, a live load (as, for example, a train) can be reduced to an equivalent load of masonry,—in which case the reduced-load contour would consist of a line $G\,H$ above and parallel to $E\,I$ for that part of the span covered by the

The value and position of the horizontal components of the external forces are somewhat indeterminate (see §§ 528–31). According to Rankine's theory of earth pressure,* the horizontal pressure of earth at any point can not be *greater* than $\frac{1 + \sin \phi}{1 - \sin \phi}$ times the vertical pressure at the same point, nor *less* than $\frac{1 - \sin \phi}{1 + \sin \phi}$ times the vertical pressure,—ϕ being the angle of repose.† If $\phi = 30°$, the above expression is equivalent to saying that the horizontal pressure can not be greater than three times the vertical pressure nor less than one third of it. Evidently the horizontal component will be greater the harder the earth spandrel-filling is rammed into place. The condition in which the earth will be deposited behind the arch can not be foretold, but it is probable that at least the minimum value, as above, will always be realized. Hence we will assume that the horizontal component is at least *one third* of the vertical pressure; that is to say, $h = \frac{1}{3} w d l$, in which w is the weight of a cubic unit of earth—which was assumed above at 100 pounds,—d the depth of the center of pressed surface below the top of the earth filling, and l the vertical dimension of the surface. The values and the positions of the horizontal forces acting on the respective sections of the arch ring are given in the second double column of Table 59.

To find the least thrust at the crown consistent with stability of rotation, assume that the center of pressure on any joint is at a distance from the intrados equal to one third of the length of the joint (see paragraph 4, page 457). The co-ordinates to the several centers of pressures are given in the third double column of Table 59. The length of the crown joint is 1.25 feet. The thrust at the crown is supposed to be applied at the upper limit of the middle third of the crown joint. Notice that the several values of x, k, and y are simply the differences between two quantities given in Table

train; while for the remainder of the span, the line IF is the reduced-load contour. The second step is to draw the arch ring and its reduced-load contour on thick paper, to a large scale, and then, with a sharp knife, carefully cut out the area representing the load on each arch stone. The center of gravity of each piece, as $i\,j\,k\,l\,m\,n$, Fig. 124, can be found by balancing it on a knife-edge; and then the position of the center of gravity is to be transferred to the drawing of the arch.

* See § 544, page 348.

† Rankine's Civil Engineering, p. 320.

59. The last three columns of the table contain the partial values and also the total values of the crown thrust as computed by equation (5), page 456.

An inspection of the results in the last column of Table 59 shows that the thrust is a maximum for joint 4. A repetition of the computations, using smaller divisions of the arch ring, might show that the absolute maximum occurs a little to one side or the other of this joint; but the uncertainty in the data for both the vertical and the horizontal forces is too great (see § 619 and §§ 527–31 respectively) to justify an attempt at absolute accuracy, and hence we will assume that joint 4 is the true joint of rupture. The angular distance of this joint from the crown is 45°, which quantity is termed the *angle of rupture.*

Any increase in the assumed intensity of the horizontal components increases the computed value of the angle of rupture. For example, if the quantities in the next to the last column of Table 59 be doubled, the thrust for joint 7 will be the maximum. Probably this condition could be realized by tightly tamping the earth spandrel-filling.

Notice that the preceding discussion of the position of the joint of rupture is for a uniform stationary load. The angle of rupture for a concentrated moving load will differ from the results found above; but the mathematical investigation of the latter case is too complicated and too uncertain to justify attempting it.

682. In discussions of the position of the joint of rupture, the horizontal components are usually neglected.* This phase of the subject will be considered only briefly. The following is the method always employed † in investigating the position of the joint of rupture, and is based on the assumption that the crown thrust is correctly given by equation (6), page 456.

Let W = the total weight resting on any joint; $\bar{x}$ = the horizontal distance of the center of gravity of this weight from the crown joint; and y = the arm of the crown thrust. Then equation (6) becomes

$$T = \frac{W\bar{x}}{y} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

* So far as observed, Rankine's investigation is the only exception; and it is, in fact, only an apparent exception (see paragraph 2, page 490).

† For example, see Sonnet's Dictionnaire des Mathématique Appliquées, pp. 1084–85.

To determine the condition for a maximum, it is assumed that W, $\bar{x}$, and y are independent variables. By differentiating equation (8), the condition for a maximum then is

$$y\,\bar{x}\frac{dW}{W}+y\,dx-\bar{x}\,dy=o. \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

The first term of equation (9) is then made equal to zero, when the condition for a maximum crown thrust becomes

$$\frac{dx}{dy}=\frac{\bar{x}}{y}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

The usual interpretation of equation (10) is: "The joint of rupture is that joint at which the tangent to the intrados passes through the intersection of T and the resultant of all the vertical forces above the joint in question."

The preceding investigation is approximate for the following reasons: 1. The effect of the horizontal forces is omitted. 2. W, $\bar{x}$, and y are dependent variables, and not independent as assumed. 3. Making the first term of equation (9) equal to zero is equivalent to assuming that the load is uniform per foot of span. 4. If the load is assumed to be uniform per foot of span, then the numerator of the second member of equation (10) should be regarded as half of the horizontal distance from the crown to the lateral limit of the load resting on the joint of rupture. 5. In the interpretation of equation (10), instead of "the tangent to the intrados," should be employed *the tangent to the line of resistance.*

In applying this method, a table, computed by M. Petit, which gives the angle of rupture in terms of the ratio of the radii of the intrados and the extrados, is generally employed. The table involves the assumption that a, Fig. 121 (p. 455), is in the extrados and b in the intrados; and also that the intrados and extrados are parallel. According to this table, "a semi-circular arch of which the thickness is uniform throughout and equal to the span divided by *seventeen and a half* is the thinnest or lightest arch that can stand. A thinner arch would be impossible." If the line of resistance is restricted to the middle third, then, according to this theory, the thinnest semi-circular arch which can stand is one whose span is *five and a half* times the uniform thickness. Many

arches in which the thickness is much less than one seventeenth of the span stand and carry heavy loads without showing any evidence of weakness. For example, in arch No. 26 of Table 63 (pp. 502–3), which is frequently cited as being a model, the average thickness is 3.25 ft., or about *one twenty-fifth* of the span; and since no joints open, the line of resistance must lie in the middle third, even though the thickness is only *one fifth* of that required by the table. Owing to the approximations involved, and also to the limitations to arches having intrados and extrados parallel, the ordinary tables for the position of the joint of rupture have little, if any, practical value. The only satisfactory way to find the angle of rupture is by trial by equation (5), as explained in § 681.

According to M. Petit's table, if the thickness is one fortieth of the diameter, the angle of rupture is 46° 12′; if the thickness is one twentieth, the angle is 53° 15′; and if one tenth, 59° 41′.

In conclusion, notice that the investigations of both this and the preceding section show that an arch of more than about 90° to 120° central angle is impossible.

683. Winkler's Hypothesis. Prof. Winkler, of Berlin,—a well-known authority—published in 1879 in the "*Zeitschrift des Architekten und Ingenieur Vereins zu Hannover*," page 199, the following theorem concerning the position of the line of resistance: "For an arch ring of constant cross section that line of resistance is approximately the true one which lies nearest to the axis of the arch ring, as determined by the method of least squares." *

The only proof of this theorem is that by it certain conclusions can be drawn from the voussoir arch which harmonize with the accepted theory of solid elastic arches. The demonstration depends upon certain assumptions and approximations, as follows: 1. It is assumed that the external forces acting on the arch are vertical; whereas in many cases, and perhaps in most, they are inclined. 2. The loads are assumed to be uniform over the entire span; whereas in many cases the arch is subject to moving concentrated loads, and sometimes the permanent load on one side of the arch is heavier than that on the other. 3. It is assumed that the load included between the lines PGD and NHC, Fig. 122 (page 458), is equal in all respects to that included between $PG2$

* This theorem was first brought to the attention of American readers in 1880, by Professor Swain in an article in Van Nostrand's Engin'g Mag., vol. xxiii, pp. 265–76.

and $NH1$. The error thus involved is inappreciable at the crown, but at the springing of semicircular arches is considerable. 4. The conclusions drawn from the voussoir (masonry) arch only approximately agree with the theory of elastic (solid iron or wood) arches. 5. Masonry arches do not ordinarily have a constant cross section as required by the above theorem; but it usually, and properly, increases toward the springing. 6. The phrase "as determined by the method of least squares" means that the true line of resistance is that for which the sum of the squares of the vertical deviations is a minimum. Since the joints must be nearly perpendicular to the line of resistance, the deviations should be measured normal to that line. For a uniform load over the entire arch, the lines of resistance are comparatively smooth curves; and hence, if the sum of the squares of the vertical deviations is a minimum, that of the normal also would probably be a minimum. But for eccentric or concentrated loads it is by no means certain that such a relation would exist. 7. The degree of approximation in this theorem is less the flatter the arch.

684. To apply the above theorem for the position of the true line of resistance, it is customary, by English writers at least, to employ the following principle, which it is asserted follows directly from Winkler's theorem: "If any line of resistance can be constructed within the middle third of the arch ring, the true line of resistance lies within the same limits, and hence the arch is stable." This assertion is disputed by Winkler himself, who says it is not, in general, correct.* It does not necessarily follow that because one line of resistance lies within the middle third of the arch ring, the "true" line of resistance also does; for the "true" line may coincide very closely with the axis in one part of the arch ring and depart considerably from it in another part, and still the sum of the squares of the deviations be a minimum. This method of applying Winkler's theorem is practically nothing more or less than an application of the conclusions derived from the hypothesis of least resistance (§ 677).

To apply Winkler's theorem without making the preceding assumption, it is necessary to (1) construct a line of resistance, (2) measure its deviations from the axis, and (3) compute the sum of

* Prof. Swain's review of Winkler's Theorem—Van Nostrand's Engin'g Mag., vol. xxiii, p. 275.

the squares of the deviations; and it is then necessary to do the same for all possible lines of resistances, the one for which the sum of the squares of the deviations is least being the "true" one.

Owing to the above defects in this hypothesis, it will not be discussed further.

685. Navier's Principle. It is well known, from the principles of fluid pressure, that the tangential force at any point of a circle pressed by normal forces is equal to the pressure per unit of area multiplied by the radius. The condition of an arch of any form at any point where the pressure is normal is similar to that of a circular arch of the same curvature under a pressure of the same intensity per unit of area; and hence the following principle: *the thrust at any normally pressed point of an arch is the product of the radius of curvature by the intensity of the pressure at that point.* Or, denoting the radius of curvature by ρ, the normal pressure per unit of length of intrados by p, and the thrust by T, we have

$$T = p\,\rho. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

This relation, due to Navier, is frequently employed by writers on the theory of the arch. Notice, however, that equation (11) is applicable only when all of the external forces are normal and equal in intensity throughout; but, as a rule, these conditions never exist with actual arches, and hence this principle is not applicable in the theory of the arch. An example of its application will be referred to later (§ 704; and 8, § 705;—pp. 482 and 486 respectively).

686. THEORIES OF THE ARCH. Various theories have been proposed from time to time, which differ greatly in the fundamental principles involved. Unfortunately, the underlying assumptions are not usually stated; and, as a rule, the theory is presented in such a way as to lead the reader to believe that each particular method "is free from any indeterminateness, and gives results easily and accurately." Every theory of the masonry arch is approximate, owing to the uncertainty concerning the amount and distribution of the external forces (§ 666), to the indeterminateness of the position of the true line of resistance (§§ 676–85), to the neglect of the influence of the adhesion of the mortar and of the elasticity of the material, and to the lack of knowledge concerning the strength of masonry; and, further, the strains in a masonry arch are indeterminate owing to the effect of variations in the material of which the

arch is composed, to the effect of imperfect workmanship in dressing and bedding the stones, to the action of the center—its rigidity, the method and rapidity of striking it,—to the spreading of the abutments, and to the settling of the foundations. These elements are indeterminate, and can never be stated accurately or adequately in a mathematical formula; and hence any theory can be at best only an approximation. The influence of a variation in any one of these factors can be approximated only by a clear comprehension of the relation which they severally bear to each other; and hence a thorough knowledge of theoretical methods is necessary for the intelligent design and construction of arches.

A few of the most important theories will now be stated, and the fundamental principles involved in each explained.

687. To save repetition, it may be mentioned here, once for all, that every theory of the arch is but a method of verification. The first step is to assume the dimensions of the arch outright, or to make them agree with some existing arch or conform to some empirical formula. The second step is to test the assumed arch by the theory, and then if the line of resistance, as determined by the theory, does not lie within the prescribed limits—usually the middle third,—the depths of the voussoirs must be altered, and the design must be tested again.

688. Rational Theory. The following method of determining the line of resistance is based upon the hypothesis of least crown thrust (§ 678), and recognizes the existence of the horizontal components of the external forces. Unfortunately, the results found by this method, as well as those by all others, are rendered somewhat uncertain by the indeterminateness of the external forces (§ 666).

689. Symmetrical Load. *General Solution.* As an example of the application of this theory, let us investigate the stability of the semi-arch shown in Fig. 125 (page 467). The first step is to determine the line of resistance. The maximum crown thrust was computed in Table 59 (page 459), as already explained (§ 681). To construct the force diagram, a line BO is drawn to scale to represent the maximum thrust as found in the fourth line of the last column of Table 59. From O, w_1 is laid off vertically upwards; and from its extremity, h_1 is laid off horizontally to the left. Then the line from O to the left-hand extremity of h_1 (not shown in this

particular case) represents the direction and amount of the external force F_1 acting upon the first division of the arch stone; and the line R_1 from B to the upper extremity of F_1 represents the resultant pressure of the first arch stone upon the one next below it. Similarly, lay off w_2 vertically upwards from the left-hand extremity of h_1, and lay off h_2 horizontally to the left; then a line F_2 from the upper end of w_2 to the left-hand end of h_2 represents the resultant of the external forces acting on the second divisions of the arch, and a line R_2 from the upper extremity of F_2 represents the resultant pressure of the second arch stone on the third. The force diagram is completed by drawing lines to represent the other values of w, h, F, and the corresponding reactions.

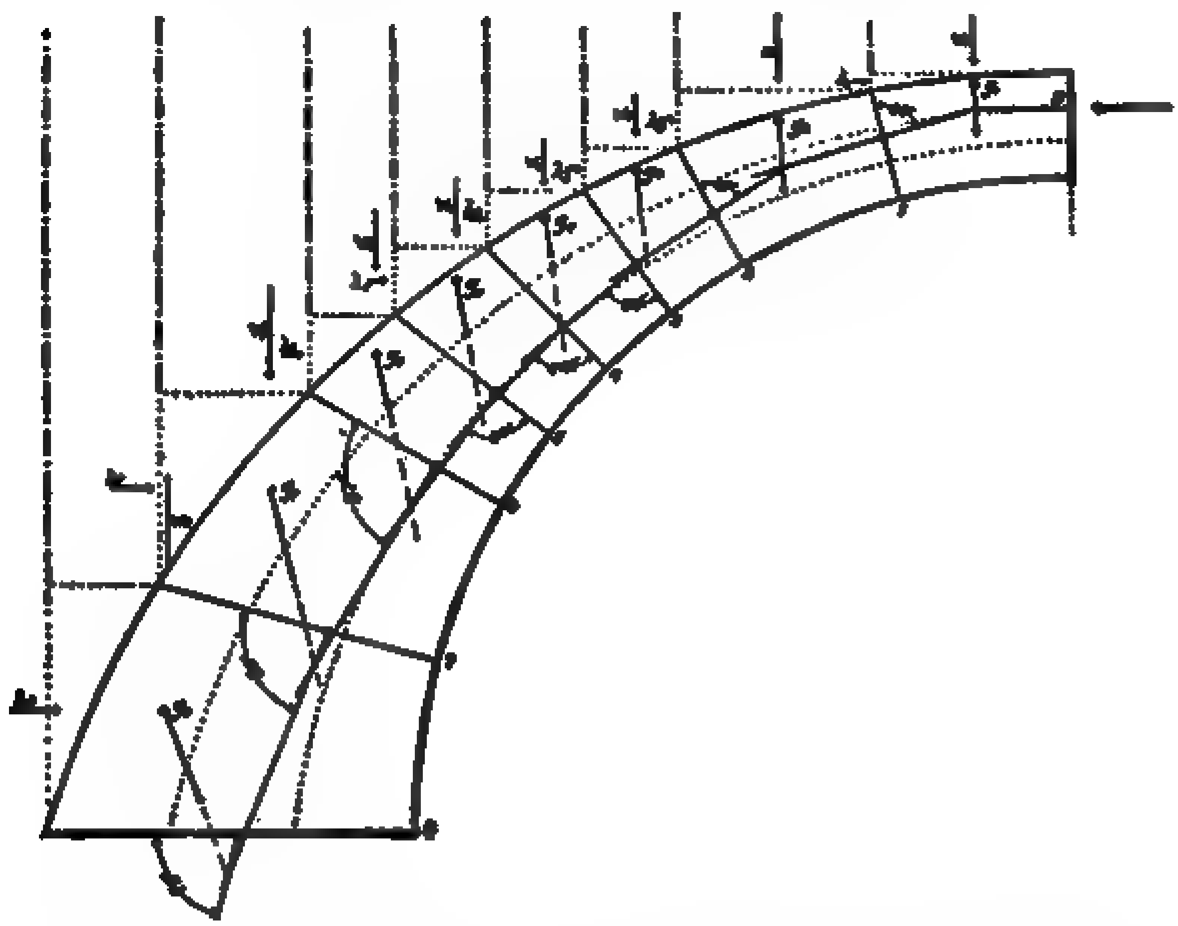

FIG. 126.

In the diagram of the arch, the points in which the horizontal and vertical forces acting upon the several arch stones intersect, are marked g_1, g_2, etc., respectively; and the oblique line through each of these points shows the direction of the resultant external force acting on each arch stone.

To construct the line of resistance, draw through U—the upper

limit of the middle third of the crown joint—a horizontal line to an intersection with the oblique force through g_1; and from this point draw a line parallel to R_1, and prolong it to an intersection with the oblique force through g_2. In a similar manner continue to the springing line. Then the intersection of the line parallel to R_1 with the first joint gives the center of pressure on that joint; and the intersection of R_2 with the second joint gives the center of pressure for that joint,—and so on for the other joints. Each center of pressure is marked by a circular dot. A line connecting these centers of pressure would be the line of resistance; but the line is not shown in Fig. 125.

690. The next step is to determine the degree of stability.

1. Since the line of resistance lies within the middle third of the arch ring, and touches the inner limit of that third at two points and its outer limit at an intermediate and higher point, the factor against rotation is 3 (see § 669).

2. The unit working pressure is found by applying equation (2), page 448. At the crown, $d = \frac{1}{6}\, l$, and hence $P = \dfrac{2\ W}{l}$; or, since $W = 9{,}400$ pounds and $l = 1.25$ feet, $P = 14{,}040$ pounds per square foot $= 98$ pounds per square inch. At the springing, $W = 21{,}700$ pounds, $l = 4.5$ feet, and $d = 0.10$ feet; and therefore

$$P = \frac{21{,}700}{4.5} + \frac{6 \times 21{,}700 \times 0.10}{(4.5)^2} = 4{,}820 + 643 = 5{,}463.$$

That is, $P = 5{,}463$ pounds per square foot, or 38 pounds per square inch. Except for a particular kind of stone and a definite quality of masonry, it is impossible even to discuss the probable factor of safety; but it is certain that in this case the nominal factor is excessive (see § 223), while the real factor is still more so (see §§ 671–72).

If the maximum pressure at the most compressed joint had been more than the safe bearing power of the masonry, it would have been necessary to increase the depth of the arch stones and repeat the entire process. Notice that the total pressure on the joints increases from the crown toward springing, and that hence the depth of the arch stones also should increase in the same direction.

3. To determine the degree of stability against sliding, notice

that the angle between the resultant pressure on any joint and the joint is least at the springing joint; and hence the stability of this joint against sliding is less than that for any other. The nominal factor of safety is equal to the co-efficient of friction divided by tan $(90° - 72°) = \tan 18° = 0.33$. An examination of Table 36 (page 315) shows that when the mortar is still wet the co-efficient is at least 0.50; and hence the nominal factor for the joint in question is at least $1\frac{1}{2}$, and probably more, while the real factor is still greater. The nominal factor for joint 7 is at least $3\frac{1}{2}$, and that for joint 3 is about 5. There is little or no probability that an arch will be found to be stable for rotation and crushing, and unstable for sliding. If such a condition should occur, the direction of the assumed joint could be changed to give stability.* The actual joints should be as nearly perpendicular to the line of resistance as is consistent with simplicity of workmanship and with stability. For circular arches, it is ordinarily sufficient to make all the joints radial. In Fig. 125, the joints are radial to the intrados; but if they had been made radial to the extrados or to an intermediate curve, the stability against sliding, particularly at the springing joint, would have been a little greater.

691. *Special Solution.* The following entirely graphical solution is useful when it is desired to find a line of resistance which will pass through two predetermined points.

For example, assume that it is desired to pass a line of resistance through U and a, Fig. 126 (page 470), the former being the upper extremity of the middle third of the crown joint and the latter the inner extremity of the middle third of joint 4.

The value and positions of the external forces, which are the same as those employed in Fig. 125, are given in Table 59 (page 459). Construct a load line, as shown in the force diagram, by laying off w_1 and h_1, and w_2 and h_2, etc., in succession, and drawing F_1, F_2, etc. Since the load is symmetrical, we may assume that the thrust at the crown is horizontal; and hence we may choose a pole at any point, say P', horizontally opposite O. Draw lines from P' to the extremities of F_1, F_2, etc. Construct a trial equilibrium polygon by drawing through U a line parallel to the line $P'O$, of the force diagram, and prolong it to b where it intersects F_1. From

* Strictly any change in the direction of the joints will necessitate a recomputation of the entire problem; but, except in extreme cases, such revision is unnecessary.

b draw a line $b\,c$ parallel to R'_1 of the force diagram; from c, the point where $b\,c$ intersects the line of F_2, draw a line $c\,d$ parallel to R'_2; from d, the point where $c\,d$ intersects F_3, draw a line $d\,e$ parallel to R'_3; and from e, the point where $d\,e$ intersects F_4, draw a line $e\,f$ parallel to R'_4. Prolong the line $f\,e$ to g, the point in

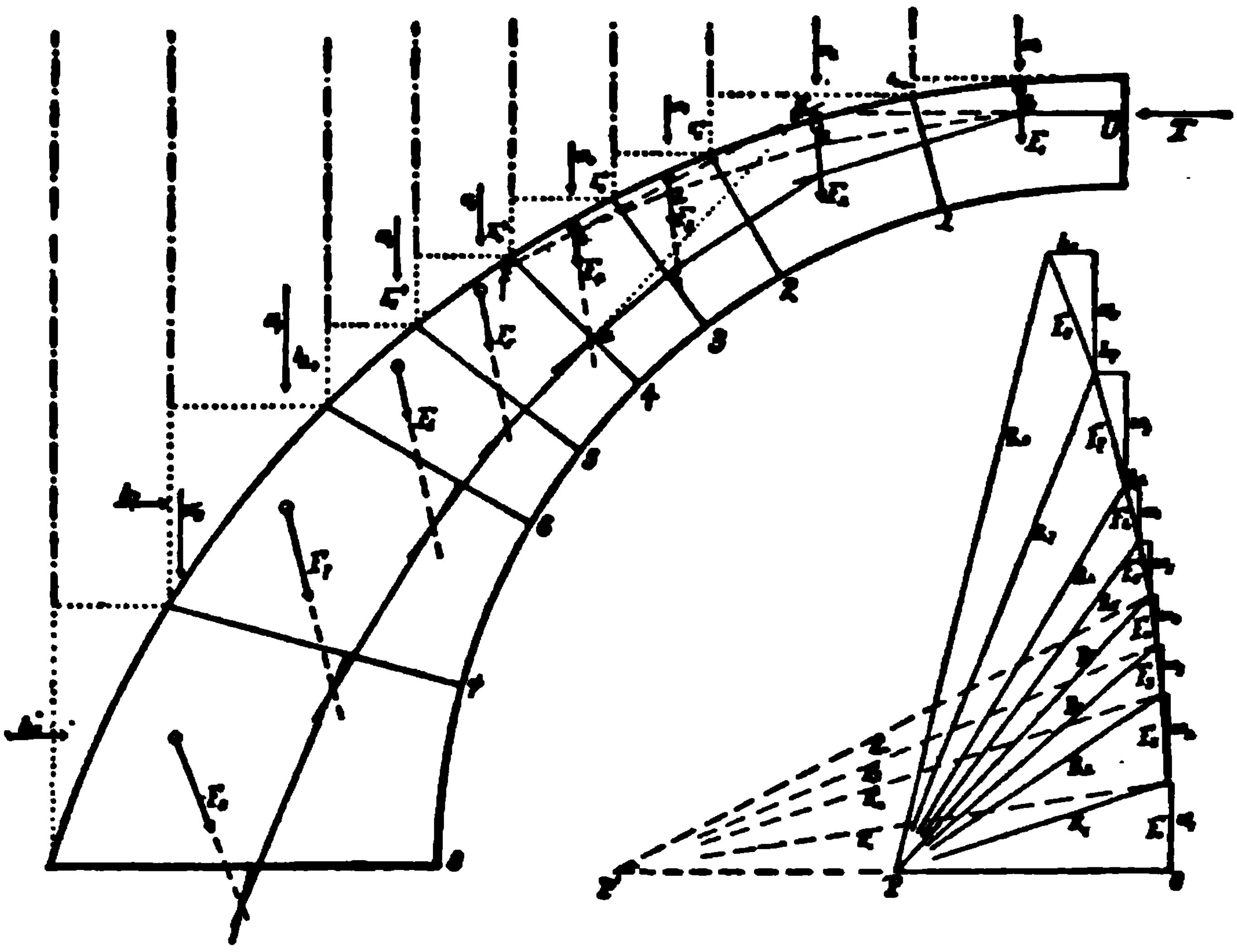

FIG. 126.

which it intersects the prolongation of $U\,b$; and then, by the principles of graphical statics, g is a point on the resultant of the forces F_1, F_2, F_3, and F_4.

The section of the arch from the crown joint to joint 4 is at rest under the action of the crown thrust T, the resultant of the external forces, and the reaction of joint 4. Since the first two intersect at g, and since it has been assumed that the center of pressure for joint 4 is at a—the inner extremity of the middle third,—a line $a\,g$ must represent the direction of the resultant reaction of joint 4; and hence the line R_4, in the force diagram drawn from the upper extremity of F_4, parallel to $a\,g$, to an intersection with $P'O$, represents, to the scale of the load line, the amount of the reaction of joint 4. Then PO, to the same scale, represents the crown thrust corresponding to the line of resistance passing through U and a; and a line—not shown in Fig. 126—from the upper

extremity of F_4 to the lower extremity of F_1, would represent, in both direction and amount, the resultant of F_1, F_2, F_3, and F_4.

Having found the thrust at the crown, complete the force diagram by drawing the lines R_1, R_2, R_3, etc.; and then construct a new equilibrium polygon exactly as was described above for the trial equilibrium polygon. The construction may be continued to the springing line. The equilibrium polygon shown in Fig. 126 by a solid line was obtained in this way.

The amount of the pressure on any joint is given by the length of the corresponding ray in the force diagram. The points in which the sides of the equilibrium polygon cut the joints are the centers of pressure on the respective joints. The stability of the arch may be discussed as in § 690.

692. One of the most useful applications of the method described in the preceding section is in determining the line of resistance for a segmental arch having a central angle so small as to make it obvious that the joint of rupture (§§ 680–81) is at the springing.

For example, assume that it is required to draw the line of resistance for the circular arch shown in Fig. 127 (p. 472). The span is 50 feet, the rise 10 feet, the depth of voussoirs 2.5 feet, and the height of the earth above the summit of the arch ring is 10 feet. The angular distance of the springing from the crown is 43° 45′; and since the angle of rupture is nearly always more than 45°, it is safe to assume that the joint of rupture is at the springing.

The method of determining the line of resistance is the same as that explained in § 691, and is sufficiently apparent from an inspection of Fig. 127.

693. Unsymmetrical Load. The design for an arch ring should not be considered perfect until it is found that the criteria of safety (§§ 668–75) are satisfied for the dead load and also for every possible position of the live load. A direct determination of the line of resistance for an arch under an unsymmetrical load is impossible. To find the line of resistance for an arch under a symmetrical load, it was necessary to make some assumption concerning (1) the amount of the thrust, (2) its point of application, and (3) its direction; but when the load is unsymmetrical, we neither know any of these items nor can make any reasonable hypothesis by which they can be determined. For an unsymmetrical load we know nothing concerning the position of the joint of

rupture, and know that the thrust at the crown is neither horizontal nor applied at one third of the depth of that joint from the

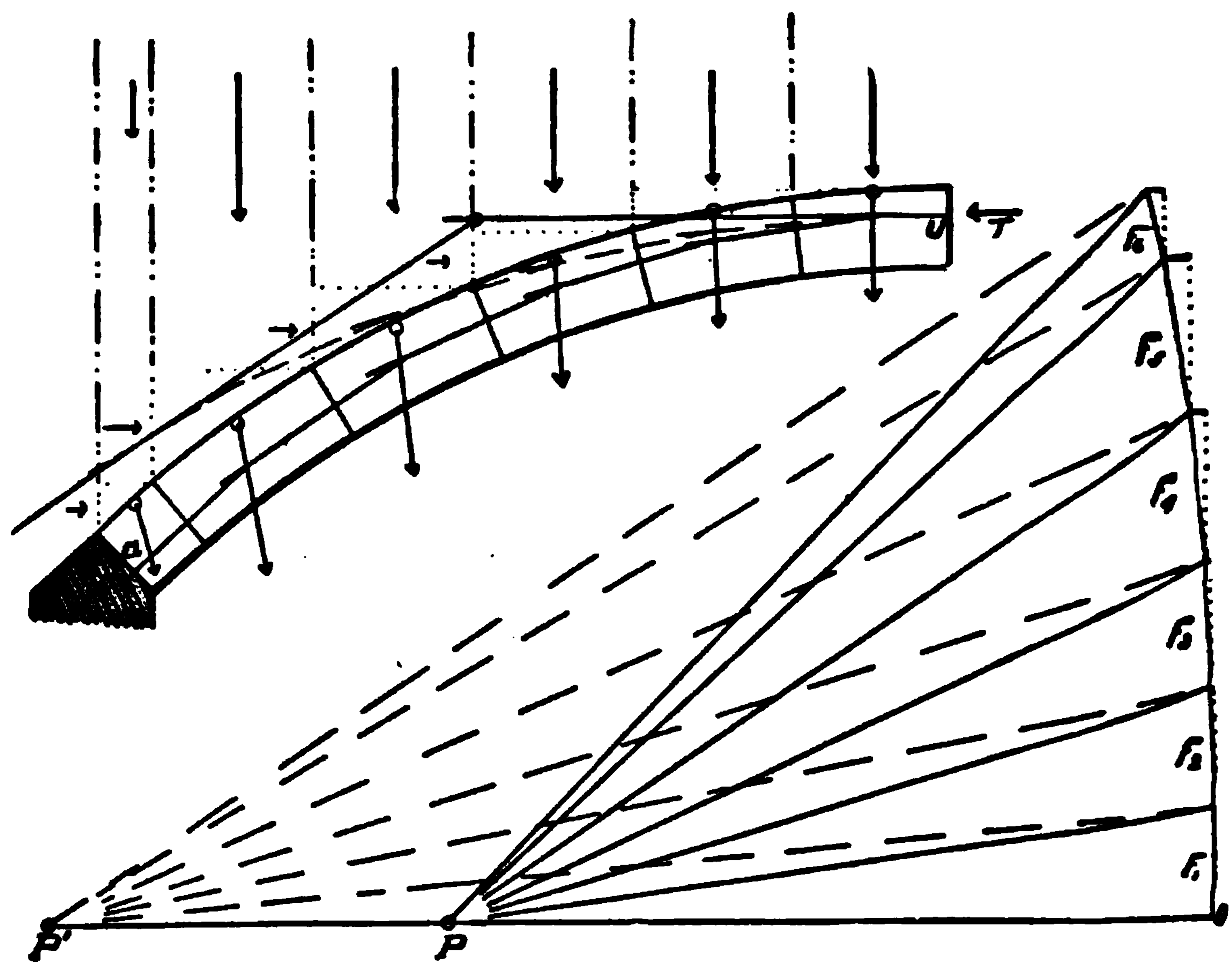

FIG. 127.

crown; and hence the preceding methods can not be employed. When the load is not symmetrical, the following method may be employed to find a line of resistance; but it gives no indication as to which of the many possible lines of resistance is the true one.

Let it be required to test the stability of a symmetrical arch having a uniform live load covering half the span. Divide the arch and its load into sections, as shown in Fig. 128. The live load is a vertical force, and the earth pressure would give a horizontal component. The approximate reduced-load contour for the vertical forces is shown in Fig. 128, and the horizontal and vertical components are laid off in the force diagram. An equilibrium polygon can be made to pass through any three points; and therefore we may assume three points for a trial equilibrium polygon,—as, for example, (1) the lower limit of the middle third of the joint at the abutment A, (2) the middle, C, of the crown joint, and (3) the upper limit of the middle third of the joint at B.

Construct a force diagram by laying off the external forces successively from O in the usual way (§ 689), selecting a pole, P', at any point, and drawing lines connecting P' with the points of division of the load line. Then, commencing at A, construct an equilibrium polygon through A, C', and B', by the method explained in §§ 691–92.

It is then necessary to move the pole of the force diagram in such a way that the equilibrium polygon will pass through B instead of B'. To do this, draw a line through the pole P', parallel to $A\,B'$—the closing line of the trial equilibrium polygon,—and then through H—the intersection of the preceding line with the load line—draw HP parallel to AB. The new pole, P, is at a point

FIG. 128.

on this line such that HP IS TO the horizontal distance from P' to the load line AS $C'D'$ IS TO CD. From P draw lines to the points of division of the load line, and then construct an equilibrium polygon through A, C, and B. If the resulting line of resistance does not lie within the middle third, try some other position of the three points A, C, and B instead of as above. If a line of resistance can not be drawn (see § 694) within the prescribed limits, then the section of the arch ring must be changed so as to include the line of resistance within the limits.

694. Criterion. If the line of resistance, when constructed by any of the preceding methods, does not lie within the middle third of the arch ring, the following process may be employed to determine whether it is possible, or not, to draw a line of resistance in the middle third.

Assume, for example, that the line of resistance of Fig. 129 lies

outside of the middle third at a and b. Next draw a line of resistance through c and d, the points where normals from a and b intersect the outer and inner boundary of the middle third respectively. To pass a line of resistance through c and d, it is necessary to determine the value and point of application of the corresponding crown thrust. The condition which makes the line of resistance pass through c is: the thrust MULTIPLIED BY the vertical distance of its point of application above c IS EQUAL TO the load on the joint at c MULTIPLIED BY its horizontal distance from c. The condition that makes the line of resistance pass through d is: the thrust MULTIPLIED BY the sum of the distance its point of application is above c and of the vertical distance between c and d IS EQUAL TO the load on the joint at d MULTIPLIED BY its horizontal distance from d. These conditions give two equations which contain two unknown quantities—the thrust and the distance its point of application is above c. After solving these equations, the line of resistance can be drawn by any of the methods already explained.

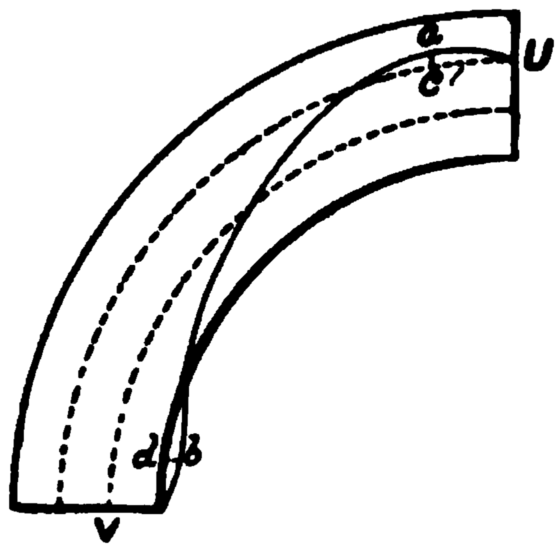

FIG. 129.

If this new line of resistance lies entirely within the prescribed limits, it is plain that it is possible to draw a line of resistance therein; but if the second line does not lie within the prescribed limits, it is not at all probable that a line of resistance can be drawn therein. The possibility of finding, by a third or subsequent trial, a line of resistance within the limits can not, in general, be answered definitely, since such a possibility depends upon the form of the section of the arch ring.

If the line of resistance drawn through U and V goes outside of the arch ring beyond the extrados only, as at a, the second line of resistance should be drawn through c and V; and if, on the other hand, it goes outside below the intrados only, as at b, the second line should be drawn through U and d.

695. SCHEFFLER'S THEORY.* This theory is the one most frequently employed. It is based upon the hypothesis of least crown thrust (§§ 678–82), and assumes that the external forces are vertical.

* See the second foot-note page 455.

This theory is frequently referred to as assuming that the arch stones are incompressible; but, fairly considered, such is not the case. Dr. Scheffler develops the theory of the position of the line of pressures for incompressible voussoirs; but subsequently states that the compressibility of the arch stones causes the line of resistance to retreat within the arch ring at points where it would otherwise reach the edge. He also says that, if a line of resistance can be drawn within the arch ring, that nowhere approaches nearer the edges of the joint than one fourth of its depth, the stability of the arch is assured.

This theory will be illustrated by two examples.

696. First Example. Assume that it is required to determine, in accordance with this theory, the line of resistance for the circular segmental arch shown in Fig. 130. The span is 50 feet, and the

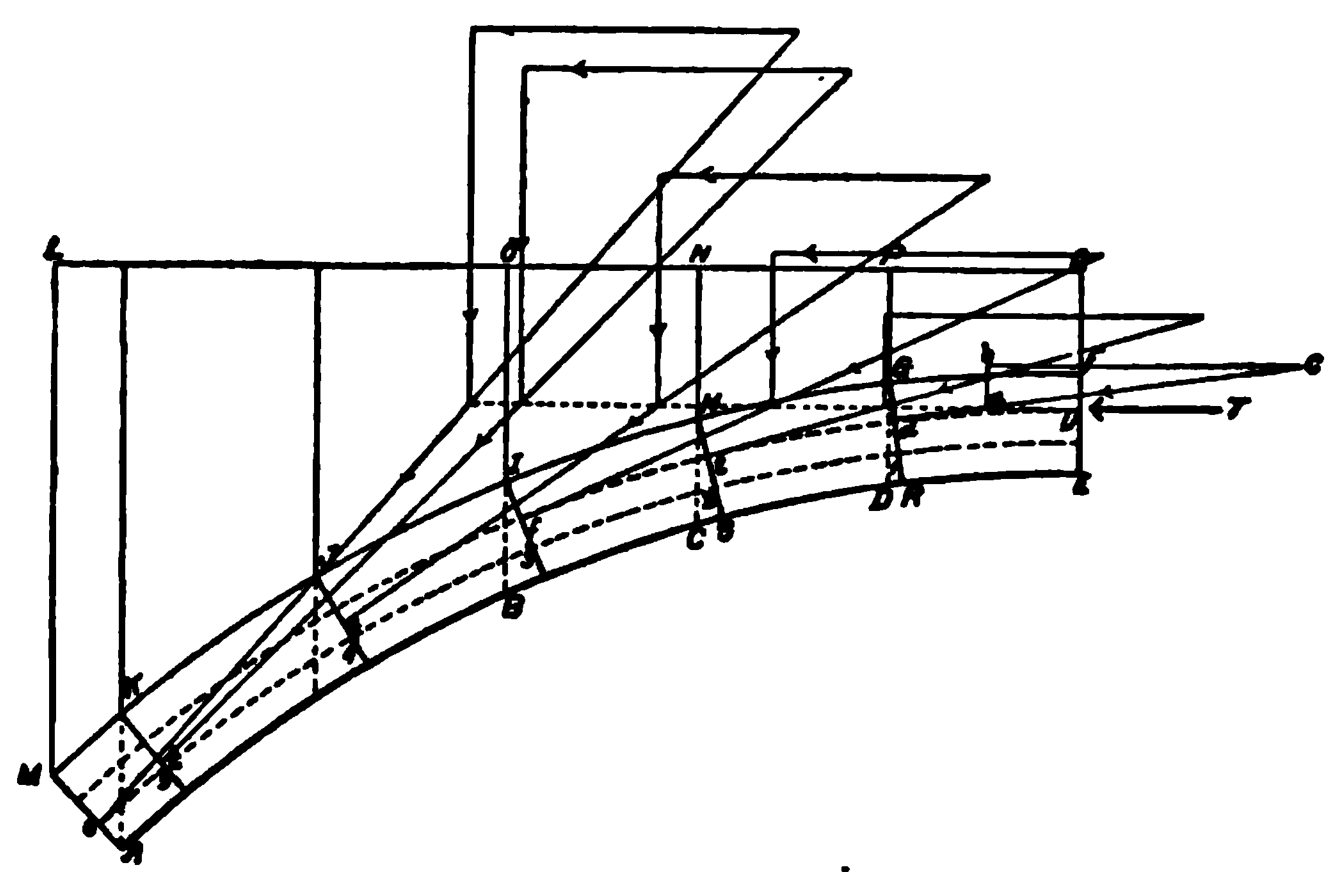

FIG. 130.

rise is 10 feet. The voussoirs are 2 feet 6 inches deep, and the spandrel wall rises 2 feet 10 inches above the summit of the arch ring. In this example we will follow the explanation used by Scheffler.*

The first step is to find the amount and the point of application of the resultant of the external forces acting on the portion of the arch above the successive joints. Divide the semi-arch and the spandrel wall into any convenient number of parts by vertical lines

* Cain's "Practical Theory of the Arch," pp. 38–44.

through F, G, H, I, J, and K, as shown. The positions of the actual joints are assumed to be not yet fixed; but, for temporary purposes, assume radial joints to be drawn through F, G, H, I, J, and K. Then the load on any part of the arch is assumed to be proportional to the area above it,—for example, the load on $CHGR$ is assumed to be proportional to the area $CNPR$.*

Having determined the area representing the loads, it is then necessary to determine (1) the numerical values of the several loads and the distances of their centers of gravity from a vertical through the crown, and (2) the amount and the position of the center of gravity of the loads above any joint. The steps necessary for this are given in Table 60.

The quantities in column 2 of Table 60 are the lengths of the medial lines of the several trapezoids. Column 6 contains the

* Notice that really the load on the joint SH, for example, is $SHNPGR$, and not $CNPR$ as above. The error is least near the crown of flat segmental arches, and greatest near the springing of semi-circular ones. The error could be eliminated (1) by finding the weights of $GPNH$ and $RGHS$ separately and combining them into a single resultant for the weight on the joint SH, as was done in § 681; or (2) by drawing the arch to a large scale on thick paper and cutting out the several six-sided figures which represent the loads, when the amounts of the several loads can be determined readily from the weights of corresponding sections of the paper, and the center of gravity of each section can be found by balancing it on a knife edge.

Scheffler gives the following empirical and approximate method of altering the position of the joints to correct this error. Let DCG, Fig. 131, be the side of the trapezoid, and CH the uncorrected joint. From b, the middle point of GH, draw

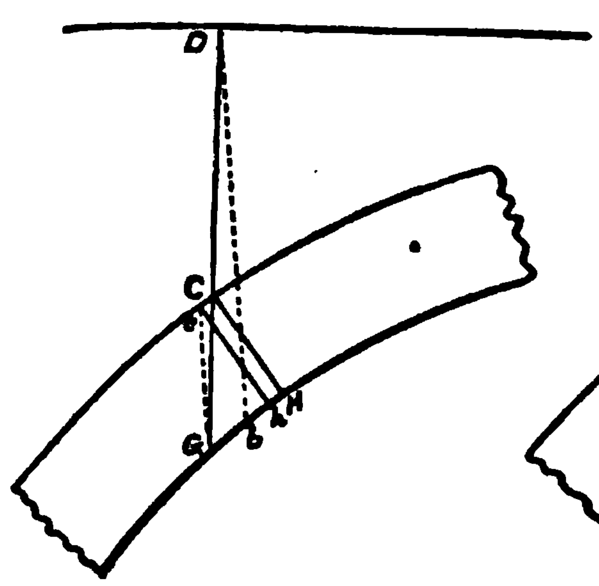

FIG. 131.

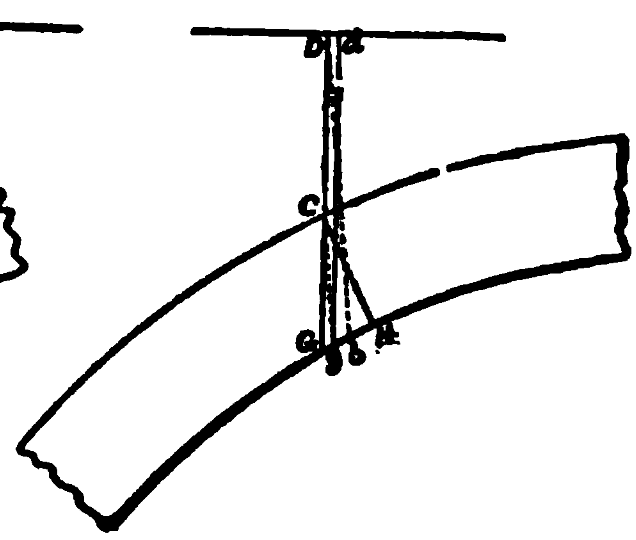

FIG. 132.

bD; and draw Gc parallel to bD, and ch parallel to CH. Then will ch be the corrected joint. Conversely, having given the joint CH, Fig. 132, to find the side of the trapezoid which limits the portion of the load upon it, through C draw DG vertical, and draw Cg parallel to Db (b being the middle point of GH); then, from g, draw dg vertical, and we have the desired side of the trapezoid.

TABLE 60.

APPLICATION OF SCHEFFLER'S THEORY TO THE ARCH RING SHOWN IN FIG. 180, PAGE 475.

1	2	3	4	5	6	7	8	9
No. of the Division of the Arch Ring, Counting from the Crown.	The Amount, and Position of the Center of Gravity, of the Several Loads.				To find the Amount, and the Center of Gravity, of the Loads above the Several Joints.			
	Dimensions of the sections.			Horizontal distance of center of gravity of each section from U.	Moment of each section about U.	Area of the load above the successive joints.	Moments of the loads above the successive joints, about U.	Horizontal distance from U to the center of gravity of the loads above the successive joints.
	Height.	Width.	Area.					
1	5.4	5	27.0	2.5	67.50	27.0	67.50	2.5
2	6.1	5	30.5	7.5	228.75	57.5	296.25	5.1
3	7.6	5	38.0	12.5	475.00	95.5	771.25	8.1
4	9.8	5	49.0	17.5	857.50	144.5	1,628.75	11.3
5	13.2	5	66.0	22.5	1,485.00	210.5	3,113.75	14.7
6	14.5	1.75	25.4	25.9	657.86	235.9	3,771.61	16.0

products of the numbers in columns 4 and 5. Column 7 contains the continued sums of the quantities in column 4. Column 8 contains the continued sums of the quantities in column 6. Column 9 is found by the principle of analytical mechanics: the distance of the center of parallel forces from any point is equal to *the sum* of the moments of the several forces about that point *divided by* the sum of the several forces; and hence the numbers in column 9 are found by dividing the quantities in column 8 by the corresponding quantity in column 7.

697. The second step is to find the minimum thrust which applied at U ($UF = \frac{1}{2} FE$) is sufficient to prevent the semi-arch from rotating. The origin of moments is considered as being in the successive joints at one third of the depth of each from the intrados.

If $T =$ the thrust and $y =$ its arms, and $W =$ the load above any joint and $x =$ its arm, then for equilibrium about any joint

$$T = \frac{Wx}{y}. \quad \dots\dots\dots \quad (12)$$

It is required to find the maximum value of T.

The W—in terms of the weight of a cubic foot of the masonry—for each joint is the corresponding number in column 7 of Table 60, and is for convenience repeated in column 2 of the table below. The x for each joint is the horizontal distance between the resultant of the load above each joint and the center of that joint; and is equal to the horizontal distance from U to the points 1, 2, etc., *minus* the respective quantities in column 9 of Table 60. The first of these quantities is given in column 3 of Table 61, the second in column 4, and their difference in column 5. The y for each joint is given in column 6 of Table 61. The value of the thrust, obtained by substituting the above data successively in equation (12) and solving, is given in column 7 of Table 61.

TABLE 61.

APPLICATION OF SCHEFFLER'S THEORY TO THE ARCH RING SHOWN IN FIG. 130, PAGE 475.

1	2	3	4	5	6	7
No. of the Joint, counting from the one next to the Crown.	Area of the load above each joint (= W).	Horizontal distance from U to 1, 2, 3, etc., respectively.	Horizontal distance from U to the center of gravity of the loads above the successive joints.	Arm of the load about the center of resistance of the successive joints (= x).	Arm of the thrust about the center of resistance of each joint (= y)	Horizontal thrust required to prevent rotation about the successive joints (= T).
1	27.0	4.8	2.5	2.3	1.15	54.0
2	57.5	9.6	5.1	4.5	2.09	123.6
3	95.5	14.4	8.1	6.3	3.72	116.9
4	144.5	19.2	11.3	7.9	6.16	185.3
5	210.5	24.0	14.7	9.3	9.60	204.0
6	235.9	25.6	16.0	9.6	11.00	205.9

The horizontal thrust for joint 6 is the greatest, and hence that joint is the joint of rupture. This result might have been anticipated, since the angle of rupture ordinarily varies between 45° and 60° (see last paragraph of § 682, page 463), while the angular distance of joint 6 from the crown is only 43° 35′.

698. The second step is to construct the line of resistance.

To find the center of pressure on joint 1, Fig. 130, page 475, draw a horizontal line through U, and lay off, to any convenient scale, a distance Ua to the left equal to the first quantity in column 4 of Table 61. a is a point through which the weight of $DEQP$*

* Assumed to be equal to $REQPG$ (see foot-note, page 476).

acts. Lay off, vertically, a distance ab equal to the first quantity in column 2 of Table 61; this line represents the weight of the first voussoir and the load resting upon it. From b lay off, horizontally to the right, a distance bc equal to the last quantity in column 7 of Table 61. This line represents the horizontal pressure at the crown. Then, by the principle of the triangle of forces, a line ca represents the resultant pressure on the joint RG; and this line prolonged intersects the joint RG at d, which is, therefore, the center of pressure on that joint.

To find the center of pressure on the second joint, lay off from U, horizontally to the left, a distance equal to the second quantity in column 4 of Table 61; erect a vertical equal to the second quantity in column 2; and from the point thus found lay off, horizontally to the right, a quantity equal to the last quantity in column 7. Then draw the third side of the triangle of forces, and prolong it until it intersects the joint at e.

By a similar construction, the centers of pressure for the several joints are determined to be U, d, e, f, g, h, and 6, as shown in Fig. 130. A line joining these points is the line of resistance (not shown in the figure).

699. The preceding method of drawing the line of resistance has two advantages: (1) The center of pressure on any joint may be found at once; and (2) any small error in draughting is confined to the joint where it first occurs. Notice, however, that the method is applicable only when the horizontal component of the pressure on the several joints is constant; that is, this method is applicable only when the external forces are assumed to be vertical.

Having determined the line of resistance by the above method, the stability of the arch can be discussed as described in § 690.

700. Second Example. Let us construct, according to this theory, the line of resistance for the semi-arch shown in Fig. 133, page 480, which is the same one discussed in § 681, where it was shown that joint 4 is the joint of rupture, and that, if the horizontal forces be disregarded, the maximum crown thrust is 8,748 pounds (see Table 59, page 459).

The crown thrust is laid off, to any convenient scale, from S to O; and the loads as given in Table 59 are laid off, to the same scale, successively from O downwards. The remainder of the

construction—shown by dash lines—is exactly similar to that described in § 689 in connection with Fig. 125, page 467.

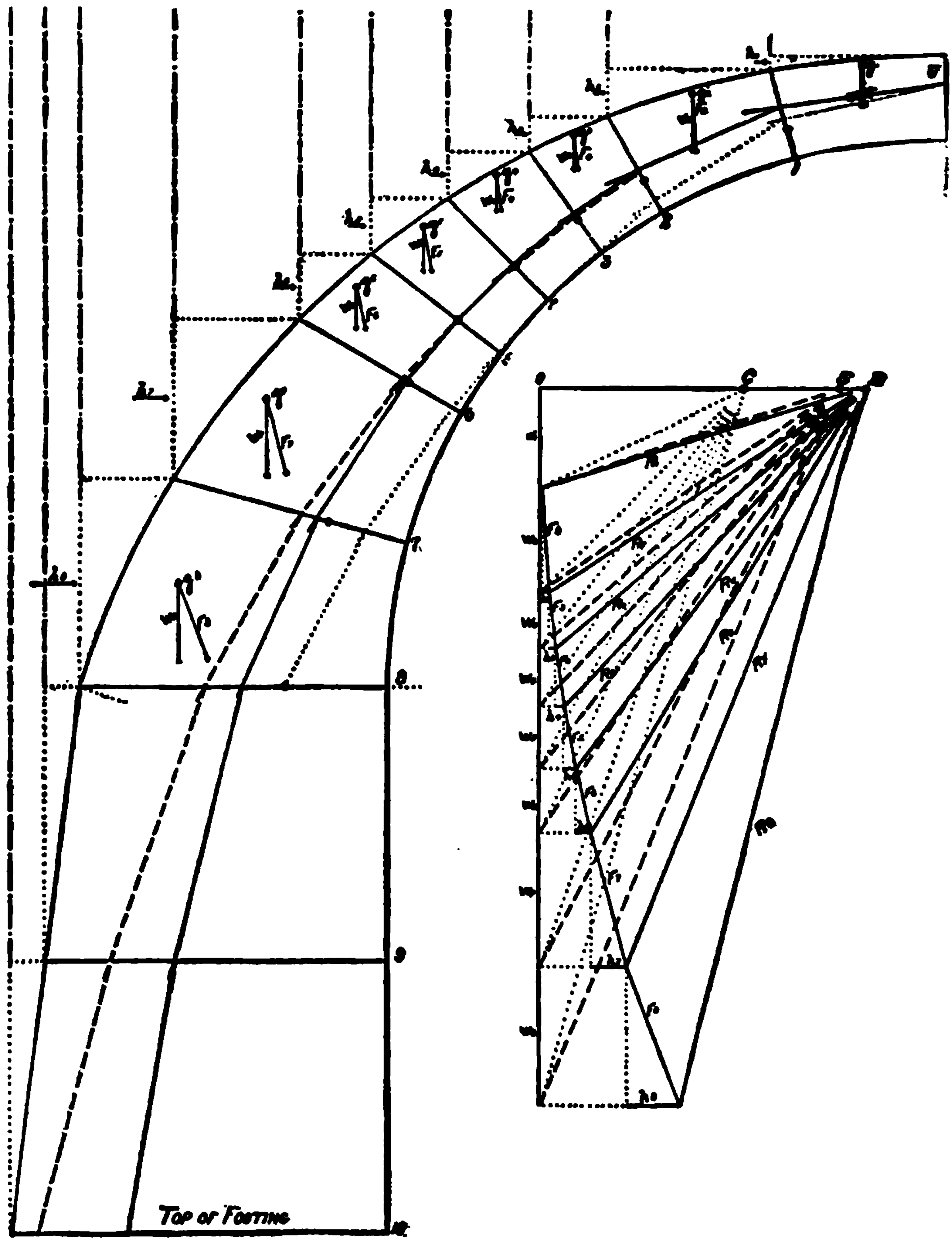

FIG. 133.

701. Erroneous Application. Frequently the principle of the joint of rupture is entirely and improperly neglected in applying this theory; that is to say, the crown thrust employed in determin-

ing the line of resistance is that which would produce equilibrium of rotation about the *springing line*, instead of that which would produce equilibrium about the *joint of rupture*. For example, instead of employing the *maximum* value in the $\frac{\Sigma w x}{y}$ column of Table 59, page 459, the *last* quantity in that column is used.

The line of resistance obtained by this method is shown in Fig. 133 (page 480) by the dotted line, the crown thrust (5,990, as computed in Table 59, page 459) being laid off from C to O, to the scale employed in laying off the load line.

702. The error of this method is shown, incidentally, in §§ 678–82 and §§ 688–701, and needs no further explanation.

The amount of the error is illustrated in Fig. 133. According to this analysis, the line of resistance is tangent to the intrados, which seems to show that the arch can not stand for a moment. However, many such arches do stand, and carry a heavy railroad traffic without any signs of weakness; and further, any reasonable method of analysis shows that the arch is not only safe, but even extravagantly so (§ 690).

This method of analysis certainly accounts for some, and perhaps many, of the excessively heavy arches built in the past. For example, compare 8 and 9, 17 and 18, 33 and 34, 52 and 54, etc., of Table 63 (page 502).

703. Reliability of Scheffler's Theory. For the sake of comparisons, the line of resistance according to the Rational Theory (§§ 688–94), as determined in Fig. 125 (page 467), is shown in Fig. 133 by the solid lines. (Notice that Fig. 133 gives the lines of resistance, and not the equilibrium polygons as in Fig. 125.) In this particular case, the difference between the two lines above the joint of rupture is not material; but the difference below that joint has a very important effect upon the thickness of the arch at the springing, and also upon the thickness of the abutment (§ 712).

If the maximum ratio of the horizontal to the vertical component of the external forces (see first paragraph on page 460) had been employed in determining the crown thrust and the line of resistance, there would have been a material difference in the position of both the joint of rupture and the line of resistance above that joint. Although the horizontal components of the external forces can not be accurately determined, any theory that disregards

the existence of these forces can not be considered more than a loose approximation.

704. Rankine's Theory. Although this theory has long been before the public and is in some respects much superior to the one in common use, it is comparatively but little employed in practice. This is probably due, in part at least, to the fact that Rankine's discussion of the theory of the masonry arch is not very simple nor very clearly stated, besides being distributed throughout various parts of his works.*

Rankine determines the thrust at the crown by Navier's principle (§ 685); but he makes no special assumption as to the point of application of this thrust, and virtually assumes that if a line of resistance can be drawn anywhere within the middle third of the arch ring, the arch is stable.

In that part of his books which precedes the discussion of arches, Rankine investigates the various curves which a cord will assume under different distributions of the load; and subsequently adopts these curves as the form which the line of resistance of an arch similarly loaded should have. The discussion of these curves constitutes the most valuable part of his investigations concerning the stability of the masonry arch.

705. Curvature of the Linear Arch. The curves assumed by a cord under the various conditions of loading, can be applied to linear arches (the line of resistance of actual arches) by imagining that the curve of the cord is reversed, and that the cord itself is replaced by a thin metal strip, which, like the cord, shall be practically without transverse strength, but which, unlike the cord, shall be able at every point to resist a compressive force in the direction of its length. The amount and distribution of the external forces are the same in both cases; but with the cord they act outward, while with the linear arch they act inward. The formulas and diagrams are essentially the same in both cases. The curves assumed by a suspended cord under various distributions of the load will now be briefly considered. In each case it will be assumed that the ends of the suspended cord and also of the corresponding linear arch are in the same horizontal line.

I. If the cord is acted upon by vertical loads distributed uni-

* "Civil Engineering," and "Applied Mechanics."

formly along the horizontal, it will assume the form of a *parabola.* This case does not occur with masonry arches.

2. If the load is vertical and distributed uniformly along the curve, the resulting curve is the *common catenary,* of which the equation is

$$y = \frac{m}{2}\left(E^{\frac{x}{m}} + E^{-\frac{x}{m}}\right), \quad . \quad . \quad . \quad . \quad . \quad (13)$$

in which y is the ordinate to any point, m the ordinate to the apex, E the base of the Naperian logarithms, and x the abscissa corresponding to y. Approximately, this case may occur with masonry arches, since the above law of loading is nearly that of an arch whose intrados is the common catenary and which supports a spandrel wall of masonry having a horizontal upper surface (see 2, page 445).

3. Three points fix the common catenary; and hence, if the position of the springing lines and the crown are assumed, the depth of the load at the crown is fixed by the equation of the curve. This limitation would often interfere with the use of the common catenary in building arches. To meet this difficulty, Rankine transforms the common catenary by the principle of what he calls parallel projections, *i. e.,* by increasing or decreasing one set of the rectangular co-ordinates to the curve without changing the other, and obtains the *transformed catenary.* The equation of the curve is

$$y = \frac{y_0}{2}\left\{E^{\frac{x}{m}} + E^{-\frac{x}{m}}\right\}, \quad . \quad . \quad . \quad . \quad . \quad (14)$$

in which y_0 is the ordinate to the apex, and m is the modulus of the curve and is found by the formula

$$m = \frac{x}{\text{hyp. log.}\left(\frac{y}{y_0} + \sqrt{\frac{y^2}{y_0^2} - 1}\right)}. \quad . \quad . \quad . \quad (15)$$

The determination of values of y by equation (14) is not easy except with either a table of Naperian logarithms or a table of results deduced therefrom, and even then it is tedious.

With this curve we may assume the springing lines, the crown, and the depth of load at the crown, and then compute the curve of equilibrium. The transformed catenary differs from a circular arc between the same points only in being slightly (and frequently only

very slightly) sharper in the haunches; and hence it is not necessary to discuss it further.*

4. If the load is uniform and normal at every point, the curve of equilibrium is plainly a *circle*. An example of this case would be an empty masonry shaft standing in water.

5. The *ellipse* is the form assumed by a cord under a load composed of horizontal and vertical components which are constant along the horizontal and vertical lines, but which differ from each other in intensity. There is no case in ordinary practice where the pressures upon an arch are strictly identical with those which give an elliptical curve of equilibrium. The curve of equilibrium of a tunnel arch through earth, when the depth below the surface is great compared with the rise of the arch itself, approximates to an ellipse. The load is nearly uniform along the horizontal, while the horizontal force at any point is some fractional part of the vertical one at the same point; and therefore the horizontal forces are nearly uniform. It is readily shown that the intensity (the pressure per unit of area perpendicular to the force) of the vertical component *is to* that of the horizontal component *as* the square of the horizontal diameter of the ellipse *is to* the square of its vertical diameter; † that is to say,

$$\frac{\text{the horizontal axis}}{\text{the vertical axis}} = \sqrt{\frac{\text{intensity of vertical component}}{\text{intensity of horizontal component}}}. \quad (16)$$

6. If the forces acting on the linear arch are normal and increase in intensity in proportion to the distance of the points of application from a horizontal line, the curve is a ***hydrostatic arch***. A tunnel under water is an example of this method of loading. The form of the curve is shown in Fig. 134, of which only the portion *BAC* is available in the construction of arches. The equation of the curve is

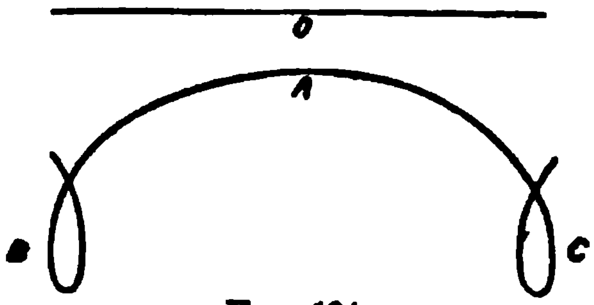

FIG. 134.

$$p\,\rho = w\,p_0\,\rho_0 = a \textit{ constant}, \quad . \ (17)$$

in which p is the normal pressure on a unit area at any point, ρ the radius of

* For two numerical examples of the method of employing the transformed catenary in the design of an arch, see an article by W. H. Booth in Van Nostrand's Engin'g Mag., vol. xxxi, pp. 1-10; and for another, see an editorial in *Engineering News*, vol. xviii, p. 372.

† Rankine's Civil Engineering, p. 205.

curvature at the same point, y the distance from the line O (the surface) to any point, ρ_0 and y_0 the values of ρ and y for the point A, and w the weight of a unit of volume of the loading.

"The true semi-ellipse of a given span and rise differs from the hydrostatic arch by being of somewhat sharper curvature at the crown and springing and of somewhat flatter curvature at the haunches, and by enclosing a somewhat less area. The application of the hydrostatic arch to practice is founded on the fact that every arch, after having been built, subsides at the crown, and spreads, or tends to spread, at the haunches, which therefore press horizontally against the filling of the spandrels; from which it is inferred as probable that, if an arch be built of a figure suited to equilibrium under fluid pressure—*i. e.*, pressure of equal intensity in all directions,—it will spread horizontally, and compress the masonry of the spandrels until the horizontal pressure at each point becomes of equal intensity to the vertical pressure, and is therefore sufficient to keep the arch in equilibrio." *

7. If the vertical and the horizontal components of the normal force differ from each other but both vary as the distance of the point of application from a horizontal line, the curve of equilbrium is the *geostatic arch*. An arch in clean dry sand is the best example of this form of loading. The geostatic arch bears the same relation to the hydrostatic arch that the ellipse does to the circle. The geostatic curve can be produced from that of the hydrostatic curve by increasing or decreasing one set of ordinates without altering the other. If p_x be the horizontal intensity of the forces acting on the hydrostatic arch and p'_x be that for the geostatic arch, then $p_x = c p'_x$; and if x is the horizontal diameter at any point of the hydrostatic curve and x' the same for the geostatic, then $x' = c\,x$.†

8. Rankine next discusses the following more general problem: "Given the curve of a linear arch and the vertical components of a symmetrical load, to find the intensity and distribution of the horizontal components necessary to produce equilibrium.

* Rankine's Civil Engineering, pp. 419–20.

† For a numerical example of the method of employing the geostatic curve for the intrados of tunnel arches, see an article—"The Employment of Mathematical Curves as the Intrados of Arches"—by W. H. Booth in Van Nostrand's Engin'g Mag., vol. xxx, pp. 355–60.

"Let V = the vertical load on any arc DC,—represented in Fig. 131 by the line EG;

V_1 = the vertical load on the semi-arch AC;

H = the horizontal load on any arc DC,—represented by the line GF, Fig. 131;

H_1 = the horizontal load on the semi-arch AC;

H_0 = the compression at the crown C,—represented by the line EC, Fig. 131;

C = the compression on the rib at any point D,—represented by ED, Fig. 131;

p_x = the intensity of the horizontal force, *i. e.*, the force per unit of area perpendicular to its line of action;

p_y = the intensity of the vertical force;

p_0 = the value of p_y at the crown C;

ρ_0 = the radius of curvature at the crown C;

i = the angle that the tangent of the linear arch at any point makes with the horizontal,—that is, i = the angle EDG, Fig. 131.

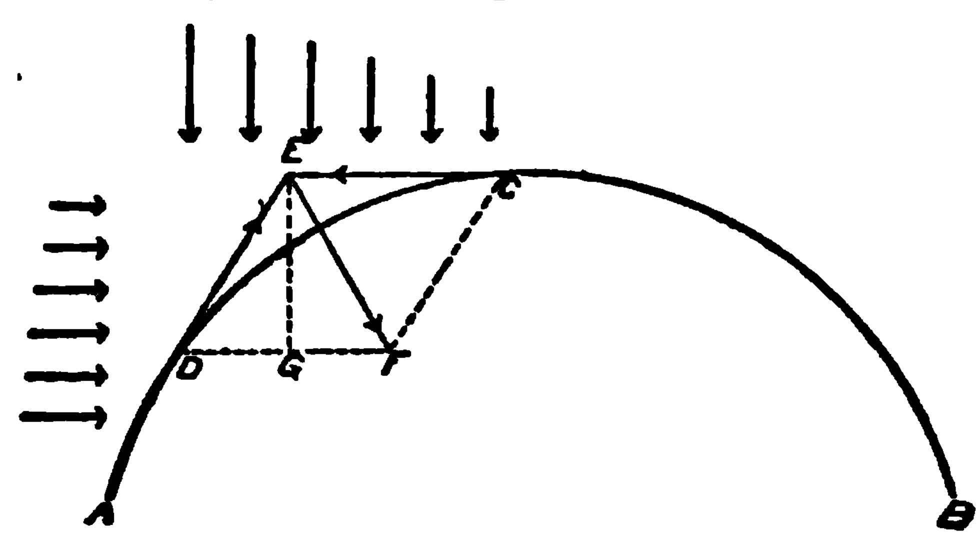

FIG. 135.

"Then
$$V = \int_0^x p_y\, dx; \qquad (18)$$
$$C = V \operatorname{cosec} i; \qquad (19)$$
$$H = V \cot i; \qquad (20)$$
$$p_x = \frac{dH}{dy} = -\frac{d(V \cot i)}{dy} = -\frac{d\left(V\frac{dx}{dy}\right)}{dy}. \qquad (21)$$

"The integration constant for (21) is H_0; and is found by equation (11), page 465, which, in the above nomenclature, becomes
$$H_0 = p_0\, \rho_0." \qquad (22)$$

However, before concluding this phase of the discussion of arches, it is well to state that the only arches in common use are the circular—either semi-circular or segmental—and the elliptic.

706. Stability of any Proposed Arch. To apply the preceding principles in designing an arch, it is necessary to know both the vertical and the horizontal forces acting on the arch. Rankine assumes* (1) that the vertical force acting on any part is the weight of the masonry, earth, or other load vertically above the same; and (2) that the horizontal pressure of earth is given by the formula

$$H = w\, d \frac{1 - \sin \phi}{1 + \sin \phi} \dagger, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (23)$$

in which H is the horizontal pressure at any point, w the weight of a unit of the earth, d the depth of earth over the point, and ϕ the angle of repose. In the above nomenclature, the vertical component is

$$V = w\, d. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

By an application of these two principles are to be determined the amount and distribution of the vertical and the horizontal forces acting on the arch; and then the equilibrium curve corresponding to this form of loading (see § 705) is to be adopted for the intrados of the proposed arch.

For an example, take the case of an arch under a high bank of earth whose angle of repose is 30°. Strictly, the curve of equilibrium is the geostatic arch (see paragraph 7, § 705); but it will be more simple and sufficiently exact, if we assume it to be an ellipse, which is equivalent to assuming that the rise of the arch is inconsiderable in comparison with the depth of earth over it. The intrados is then to be an ellipse in which

$$\frac{\textit{the horizontal axis}}{\textit{the vertical axis}} = \sqrt{\frac{V}{H}} = \sqrt{\frac{1 - \sin \phi}{1 + \sin \phi}} = \sqrt{\tfrac{1}{3}}. \quad . \quad . \quad (25)$$

"If the earth is firm, and little liable to be disturbed, the proportion of the half-span—or horizontal semi-axis—to the rise—or ver-

* Civil Engineering, p. 434.

† Rankine states (Civil Engineering, p. 320) that the horizontal pressure can not be *greater* than $w\,h \frac{1+\sin\phi}{1-\sin\phi}$, nor *less* than $w\,h \frac{1-\sin\phi}{1+\sin\phi}$. Notice that the value employed above is the minimum.

tical semi-axis—may be made *greater* than is given by the preceding equation, and the earth will still resist the additional horizontal thrust; but that proportion should never be made *less* than the value given by the equation, or the sides of the archway will be in danger of being forced inwards." *

"There are numerous cases in which the form of the linear rib suited to sustain a given load may at once be adopted for the intrados of a real arch for sustaining the same load, with sufficient exactness for practical purposes. The following is the test whether this method is applicable in any given case. Let ACB in Fig. 136 be one half of the ideal rib which it is proposed to adopt as the intrados of a real arch. Draw Aa normal to the rib at the crown, so as to represent a length not exceeding two thirds of the intended depth of the keystone. Draw a normal Bb at the springing of a length such that

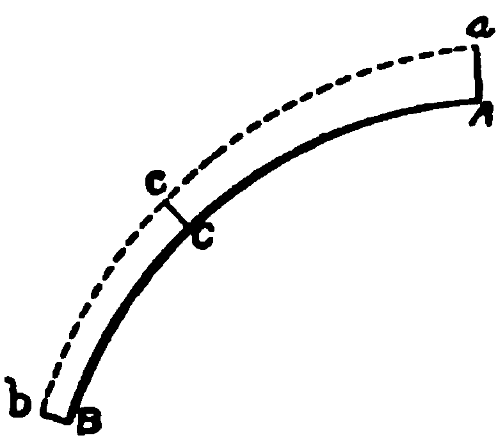

FIG. 136.

$$\frac{Bb}{Aa} = \frac{\text{thrust along rib at } A}{\text{thrust along rib at } B}.\text{"}\dagger \quad . \quad . \quad . \quad . \quad (26)$$

The thrust at A is found by equation (11), page 465. The thrust at any point, as D, is given by equation (19), page 486. Then if acb lies within the middle third of the proposed arch ring, the ideal rib ACB is of a suitable form for the intrados.

707. Rankine's general method of determining the stability of a proposed arch is as follows: ‡

"The first step towards determining whether a proposed arch will be stable, is to assume a linear arch parallel to the intrados or soffit of the proposed arch, and loaded vertically with the same weight, distributed in the same manner. Then by equation (21), page 486, determine either a general expression, or a series of values, of the intensity p_x of the conjugate pressure, horizontal or oblique as the case may be, required to keep the arch in equilibrio under the given vertical load. If that pressure is nowhere negative, a curve, similar to the assumed arch, drawn through the middle

* Rankine's Civil Engineering, p. 434.
† *Ibid.*, p. 417.
‡ *Ibid.*, pp. 421–22.

of the arch ring will be, either exactly or very nearly, the line of pressure of the proposed arch; p_x will represent, either exactly or very nearly, the intensity of the lateral pressure which the real arch, tending to spread outwards under its load, will exert at each point against its spandrel and abutments; and the thrust along the linear arch at each point will be the thrust of the real arch at the corresponding joint.

"On the other hand, if p_x has some negative values for the assumed linear arch, there must be a pair of points in that arch where that quantity changes from positive to negative, and is equal to nothing. The angle of inclination i at that point, called the *angle of rupture*, is to be determined by placing the second member of equation (21), page 486, equal to zero and solving for $\cot i$. The corresponding joints in the real arch are called the joints of rupture; and it is below those joints that conjugate pressure from without is required to sustain the arch and that consequently the backing must be built with squared side-joints.

"In Fig. 137, let BCA represent one half of a symmetrical arch, $KLDE$ an abutment, and C the joint of rupture—found by the method already described. The point of rupture, which is the center of resistance of the joint of rupture, is somewhere within the middle third of the depth of that joint; and from that point down to the springing joint B, the line of pressure is a curve similar to the assumed linear arch, and parallel to the intrados, being kept in equilibrio by the lateral pressure between the arch, and its spandrel and abutment.

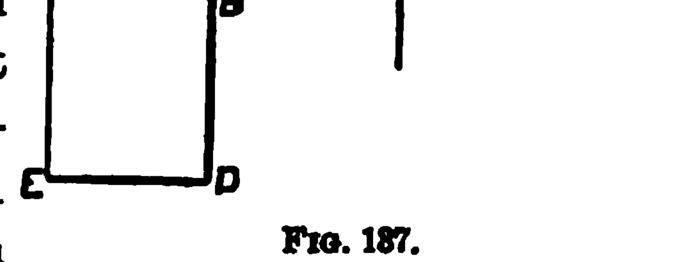

FIG. 137.

"From the joint of rupture C to the crown A, the figure of the true line of pressure is determined by the condition that it shall be a linear arch balanced under vertical forces only; * that is to say, the horizontal component of the thrust along it at each point is a

* From this it appears that Rankine himself disregards, for that part of the arch above the joint of rupture, the principal characteristic of his theory, viz.: the recognition of the horizontal components of the external forces; and hence this theory is, in fact, the same as Scheffler's (§§ 695–708).

constant quantity, and equal to the horizontal component of the thrust along the arch at the joint of rupture.

"The only point in the line of pressure above the joint of rupture which it is important to determine is that of the crown of the arch, A; and it is found in the following manner: Find the center of gravity of the load between the joint of rupture C and the crown A; and draw through that center of gravity a vertical line. Then if it be possible, from any point, such as M, in that vertical line, to draw a pair of lines, one parallel to a tangent to the soffit at the joint of rupture and the other parallel to a tangent to the soffit at the crown, so that the former of those lines shall cut the joint of rupture and the latter the keystone, in a pair of points which are both within the middle third of the depth of the arch ring, the stability of the arch will be secure; and if the first point be the point of rupture, the second will be the center of resistance at the crown of the arch and the crown of the true line of pressures.

"When the pair of points, related to each other as above, do not fall at opposite limits of the middle third of the arch ring, their exact positions are to a small extent uncertain; but that uncertainty is of no consequence in practice. Their most probable positions are equidistant from the middle line of the arch ring.

"Should the pair of points fall beyond the middle third of the arch ring, the depth of the arch stones must be increased."

708. Reliability of Rankine's Theory. Rankine's theory is approximate for the following reasons:

1. The method of deducing the thrust at the crown (see § 685) is applicable only where the external forces are normal, over the entire arch, to the line of resistance. We know that, in many practical cases, this condition does not exist; and we can not be assured that it does in any. It is certainly not correct to deduce the crown thrust under the assumption of *normal* loads, and then employ it under the assumption that the loads are *vertical*—as Rankine does.*

2. The method of finding the center of pressure at the crown and also at the joint of rupture (see the last paragraph on page 489) assumes that the portion CMA, Fig. 137, is acted upon by only three forces; viz., the vertical load, the thrust at the crown, and

* Civil Engineering, p. 418, and elsewhere.

the pressure on the joint of rupture. This is erroneous (*a*) because it neglects the horizontal components of the external forces, and hence the actual center of pressure at the joint of rupture is nearer the intrados than the position of C as found in Fig. 137; and (*b*) because it finds a new value for the thrust at the crown which, in general, will differ from that employed in finding the position of the joint of rupture.

Apparently Rankine himself does not believe much in this theory, as he deduces empirical formulas from the proportions of actual arches, which he recommends as giving minimum dimensions.

709. OTHER THEORIES OF THE ARCH. There are several methods, in more or less common use, of determining the stability of the voussoir arch, many of which are but different combinations of the preceding principles, while some have a much less satisfactory basis. It is not necessary to discuss any of these at length; but there is one which, owing to the frequency with which it is employed, requires a few words. It is the same as Scheffler's (§§ 695–703), except in assuming that the line of resistance passes through the *middle* of the crown joint and also through the *middle* of the springing joint. The line of resistance is then determined in any one of a number of ways; and the arch is said to be stable, if the line of resistance lies in the *middle third* of the section of the arch ring. This theory is much less satisfactory than Scheffler's and possesses no advantage over it.

710. THEORY OF THE ELASTIC ARCH. It has long been recognized that all theories for the voussoir arch are very unsatisfactory; and hence it has been proposed to consider the masonry arch as an elastic curved beam fixed at its ends, and examine its stability by the principles employed in computing the strains in arches of iron or wood. There is no essential difference, as far as the theory is concerned, between the iron and the stone arch; but there is great difficulty in applying the mathematical theory of elasticity to the masonry arch. The theory of elasticity when applied to the masonry arch has the following sources of error, in addition to those of the ordinary theory of the elastic arch: 1. There is great uncertainty as to the external forces (§ 666). 2. We have no definite knowledge concerning either the modulus of elasticity (§§ 16 and 146) or the ultimate strength of masonry (§§ 221–23, and §§ 246–

49). 3. The stone arch is not homogeneous; *i. e.*, the modulus of elasticity is not constant, but varies between that of the stone and the mortar. 4. Slight imperfections in the workmanship—as, for example, a projection on the bearing surface of an arch stone or a pebble in the mortar—would break the continuity of the arch, and render the theory inapplicable. 5. The stability of the arch would be greatly influenced by the action of the center,—its rigidity, the method of loading it to prevent deformation, and the method and rapidity of striking it.

The application of the theory of elasticity to stone arches has been considerably discussed in late years; but it is generally conceded that the results are, for the most part, illusory, since the much simpler methods give results equally reliable. The explanation of the theory of the elastic masonry arch as given by Professor Greene in Part III—Arches—of his "Trusses and Arches" is all that can be desired; and hence this theory will not be discussed here.

711. **Stability of Abutments and Piers.** The stability of the abutment is in a measure indeterminate, since it depends upon the position of the line of resistance of the arch. The stability of the abutment may be determined most easily by treating it as a part of the arch, *i. e.*, by extending the load line so as to include the forces acting upon it and drawing the reactions in the usual way; or its stability may be determined as follows: Assume that it is required to test the stability of the abutment shown in Fig. 138. Let qc represent the direction of the resultant pressure on the joint AB. g is the center of gravity of the section ABC of the abutment, and g_2 that for the section $ABED$.* At a—the point where a vertical through g intersects qc_1 prolonged—lay off, to scale, a line ad equal to the weight of ABC, and also a line ab equal to the pressure qc_1; then c_2—the point where the diagonal ea pierces AC—is the center of pressure on AC.

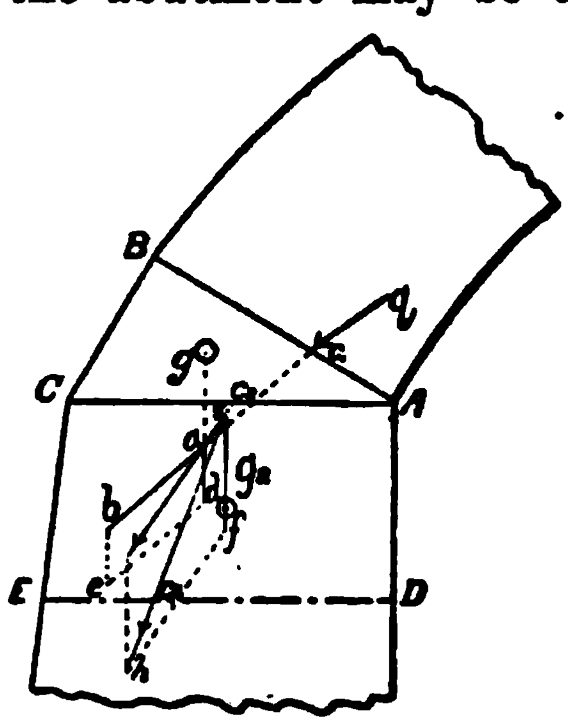

FIG. 138.

* For a method of finding the center of gravity when the section is a trapezoid, see the third paragraph of § 494 (page 318).

In a similar manner, c_3 is found to be the center of pressure on *DE*.

The amount of the pressure on *AC* is given by the length of the line *ae*; and the stability of the joint against crushing can be determined as described in §§ 670–72 and paragraph 2 of § 690. The stability against rotation may be determined as described in § 669 and paragraph 1 of § 690. A line—not shown—connecting c_1, c_2, c_3, is the line of resistance of the abutment, to which the joints should be nearly perpendicular (see § 674 and division 3 of § 690).

712. In Fig. 133 (page 480) is shown the line of resistance for the abutment according to the rational theory of the arch (§§ 688–94), and also that according to Scheffler's theory (§§ 695–703),—the former by the solid line and the latter by the broken one. Since to overestimate the horizontal components of the external forces would be to err on the side of danger, in applying the former theory in Fig. 133, the horizontal component acting against the abutment was disregarded on the assumption that the abutment might be set in a pit without greatly disturbing the surrounding earth. If the horizontal component had been considered, the difference between the lines of resistance according to the two theories would have been still greater. Notice that the analysis which recognizes the existence of the horizontal forces, *i. e.*, the rational theory, permits a lighter abutment than the theory which assumes the external forces to be entirely vertical.

The omission of the horizontal components assumes that the only object of the abutment is to resist the thrust of the arch; and that consequently the flatter the arch the greater the thrust and the heavier the abutment. Ordinarily the abutment must resist the thrust of the arch tending to overthrow it and to slide it *outward*, and must act also as a retaining wall to resist the lateral pressure of the earth tending to overthrow it and to slide it *inward*. For large arches the former is the more important; but for small arches, particularly under high embankments, the latter is the more important. Hence, for large arches or for an arch having a light surcharge, the abutment should be proportioned to resist the thrust of the arch; but for small arches under a heavy surcharge of earth, the abutment should be proportioned as a retaining wall (Chap. XIV).

Although the horizontal pressure of the earth can not be computed accurately, there are many conditions under which the horizontal components should not be omitted. For example, if the abutment is high, or if the earth is deposited artificially behind it, ordinarily it would be safe to count upon the pressure of the earth to assist in preventing the abutment from being overturned outwards. Finally, although it may not always be wise to consider the earth pressure as an active force, there is always a passive resistance which will add greatly to the stability of the abutment, and whose intensity will increase rapidly with any outward movement of the abutment (see last paragraph of § 666).

For empirical rules for the dimensions of abutments, see §§ 722–23.

Art. 2. Rules Derived from Practice.

713. In the preceding article it was shown that every theory of the arch requires certain fundamental assumptions, and that hence the best theory is only an approximation. Further, since it is practically impossible, by any theory (§ 693), to include the effect of passing loads, theoretical results are inapplicable when the moving load is heavy compared with the stationary load. It was shown also that the stability of a masonry arch does not admit of exact mathematical solution, but is to some extent an indeterminate problem. At best the strains in a masonry arch can never be computed anything like as accurately as those in metallic structures. However, this is no serious matter, since the material employed in the former is comparatively cheap.

Considered practically, the designing of a masonry arch is greatly simplified by the many examples furnished by existing structures which afford incontrovertible evidence of their stability by safely fulfilling their intended duties, to say nothing of the history of those structures which have failed and thus supplied negative evidence of great value. In designing arches, theory should be interpreted by experience; but experience should be studied by the light of the best theory available.

This article will be devoted to the presentation of current practice as shown by approved empirical formulas and practical rules, and by examples.

714. Empirical Formulas. Numerous formulas derived from existing structures have been proposed for use in designing masonry arches. Such formulas are useful as guides in assuming proportions to be tested by theory, and also as indicating what actual practice is and thus affording data by which to check the results obtained by theory.

As proof of the reliability of such formulas, they are frequently accompanied by tables showing their agreement with actual structures. Concerning this method of proof, it is necessary to notice that (1) if the structures were selected because their dimensions agreed with the formula, nothing is proven; and (2) if the structures were designed according to the formula to be tested, nothing is proven except that the formula represents practice which is probably safe.

At best, a formula derived from existing structures can only indicate safe construction, but gives no information as to the degree of safety. These formulas usually state the relation between the principal dimensions; but the stability of an arch can not be determined from the dimensions alone, for it depends upon various attendant circumstances,—as the condition of the loading (if earth, upon whether loose or compact; and if masonry, upon the bonding, the mortar, etc.), the quality of the materials and of the workmanship, the manner of constructing and striking the centers, the spreading of the abutments, the settlement of the foundations, etc. The failure of an arch is a very instructive object lesson, and should be most carefully studied, since it indicates the least degree of stability consistent with safety. Many masonry arches are excessively strong; and hence there are empirical formulas which agree with existing structures, but which differ from each other 300 or 400 per cent. All factors of the problem must be steadily borne in mind in comparing empirical formulas either with each other or with theoretical results.

A number of the more important empirical formulas will now be given, but without any attempt at comparisons, owing to the lack of space and of the necessary data.

715. Thickness of the Arch at the Crown. In designing an arch, the first step is to determine the thickness at the crown, *i. e.*, the depth of the keystone.

Let d = the depth at the crown, in feet;
ρ = the radius of curvature of the intrados, in feet;
r = the rise, in feet;
s = the span, in feet.

716. *American Practice.* Trautwine's formula for the depth of the keystone for a *first-class cut-stone* arch, whether circular or elliptical, is

$$d = \frac{\sqrt{\rho + \frac{1}{2} s}}{4} + 0.2. \quad . \quad . \quad . \quad . \quad . \quad (27)$$

"For *second-class work,* this depth may be increased about one eighth part; and for *brick work* or *fair rubble,* about one third."

717. *English Practice.* Rankine's formula for the depth of keystone for a *single* arch is

$$d = \sqrt{0.12\, \rho}\,; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (28)$$

for an arch of a *series,*

$$d = \sqrt{0.17\, \rho}\,; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (29)$$

and for *tunnel arches,* where the ground is of the firmest and safest,

$$d = \sqrt{0.12 \frac{r^2}{s}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (30)$$

and for soft and slipping materials,

$$d = \sqrt{0.48 \frac{r^2}{s}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

The segmental arches of the *Rennies* and the *Stephensons,* which are generally regarded as models, "have a thickness at the crown of from $\frac{1}{30}$ to $\frac{1}{33}$ of the span, or of from $\frac{1}{26}$ to $\frac{1}{30}$ of the radius of the intrados."

718. *French Practice.** Perronnet, a celebrated French engineer, is frequently credited with the formula,

$$d = 1\tfrac{1}{12} + \tfrac{1}{24} s, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

* From "Proportions of Arches from French Practice," by E. Sherman Gould in Van Nostrand's Engin'g Mag., vol. xxix, p. 450.

as being applicable to arches of all forms—semi-circular, segmental, elliptical, or basket-handled,—and to railroad bridges or arches sustaining heavy surcharges of earth. "Perronnet does not seem, however, to have paid much attention to the rule; but has made his bridges much lighter than the rule would require." Other formulas of the above form, but having different constants, are also frequently credited to the same authority. Evidently Perronnet varied the proportions of his arches according to the strength and weight of the material, the closeness of the joints, the quality of mortar, etc.; and hence different examples of his work give different formulas.

Dejardin's formulas, which are frequently employed by French engineers, are as follows:

For circular arches,

$$\text{if } \frac{r}{s} = 1, \quad d = 1 + 0.1\,\rho; \quad \ldots\ldots \quad (33)$$

$$\text{if } \frac{r}{s} = \tfrac{1}{4}, \quad d = 1 + 0.05\,\rho; \quad \ldots\ldots \quad (34)$$

$$\text{if } \frac{r}{s} = \tfrac{1}{8}, \quad d = 1 + 0.035\,\rho; \quad \ldots\ldots \quad (35)$$

$$\text{if } \frac{r}{s} = \tfrac{1}{16}, \quad d = 1 + 0.02\,\rho; \quad \ldots\ldots \quad (36)$$

For elliptical and basket-handled arches,

$$\text{if } \frac{r}{s} = \tfrac{1}{3}, \quad d = 1 + 0.07\,\rho. \quad \ldots\ldots \quad (37)$$

Croizette-Desnoyers, a French authority, recommends the following formulas:

$$\text{if } \frac{r}{s} > \tfrac{1}{4}, \quad d = 0.50 + 0.28\sqrt{2\,\rho}; \quad \ldots \quad (38)$$

$$\text{if } \frac{r}{s} = \tfrac{1}{4}, \quad d = 0.50 + 0.26\sqrt{2\,\rho}; \quad \ldots \quad (39)$$

$$\text{if } \frac{r}{s} = \tfrac{1}{12}, \quad d = 0.50 + 0.20\sqrt{2\,\rho}; \quad \ldots \quad (40)$$

719. Notice that in none of the above formulas does the character of the material enter as a factor. Notice also that none of them has a factor depending upon the amount of the load.

Table 62 is given to facilitate the comparisons of the preceding formulas with each other and with actual structures. Values not given in the table can be interpolated with sufficient accuracy. It is remarkable that according to all formulas credited to Perronnet the thickness at the crown is independent of the rise, and varies only with the span. Notice that by Dejardin's formulas the thickness decreases as the rise increases,—as it should.

TABLE 62.

COMPARISON OF EMPIRICAL FORMULAS FOR DEPTH OF KEYSTONE.

FORMULA.	PROPORTION OF RISE TO SPAN.								
	Semi-circle.			$\frac{\text{Rise}}{\text{Span}} = \frac{1}{4}$			$\frac{\text{Rise}}{\text{Span}} = \frac{1}{10}$		
	SPAN.			SPAN.			SPAN.		
	10	50	100	10	50	100	10	50	100
Trautwine's, for first-class work	.99	1.98	2.70	1.11	2.23	3.09	1.26	2.57	3.55
" second " "	1.11	2.23	3.04	1.25	2.51	3.43	1.44	2.89	4.00
" third " "	1.32	2.64	3.60	1.48	2.97	4.12	1.68	3.42	4.73
Rankine's	.77	1.73	2.45	1.00	2.25	3.16	1.25	2.79	3.95
Perronnet's	1.51	3.26	5.48	1.51	3.26	5.48	1.51	3.26	5.43
Dejardin's	1.50	3.50	6.00	1.42	3.07	5.17	1.26	2.30	2.60
Croizette-Desnoyers's	1.38	2.48	3.30	1.56	2.86	3.85	1.62	3.01	4.05

720. Thickness of the Arch at the Springing. Generally the thickness of the arch at the springing is found by an application of theory; and hence but few empirical formulas are given for this purpose.

Trautwine gives a formula for the thickness of the abutment, which determines also the thickness of the arch at the springing (see § 722).

"The augmentation of thickness at the springing line is made, by the Stephensons, from 20 to 30 per cent.; and by the Rennies, about 100 per cent."

721. If the loads are vertical, the horizontal component of the compression on the arch ring is constant; and hence, to have the mean pressure on the joints uniform, the vertical projection of the

joints should be constant. This principle leads to the following formula, which is frequently employed: *The length, measured radially, of each joint between the joint of rupture and the crown should be such that its vertical projection is equal to the depth of the keystone.* In algebraic language, this rule is

$$l = d \sec \alpha, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (41)$$

in which l is the length of the joint, d the depth at the crown, and α the angle the joint makes with the vertical.

The length of the joint of rupture,* *i. e.*, the thicknesss of the arch at the practical springing line, can be computed by the above formula. The following are the values for circular and segmental arches:

$$\text{If } \frac{r}{s} > \tfrac{1}{4}, \quad l = 2.00\,d; \quad . \quad . \quad . \quad . \quad . \quad (42)$$

$$\text{“ } \frac{r}{s} = \tfrac{1}{6}, \quad l = 1.40\,d; \quad . \quad . \quad . \quad . \quad . \quad (43)$$

$$\text{“ } \frac{r}{s} = \tfrac{1}{8}, \quad l = 1.24\,d; \quad . \quad . \quad . \quad . \quad . \quad (44)$$

$$\text{“ } \frac{r}{s} = \tfrac{1}{10}, \quad l = 1.15\,d; \quad . \quad . \quad . \quad . \quad . \quad (45)$$

$$\text{“ } \frac{r}{s} = \tfrac{1}{12}, \quad l = 1.10\,d. \quad . \quad . \quad . \quad . \quad . \quad (46)$$

722. Thickness of the Abutment.† *Trautwine's* formula is

$$t = 0.2\,\rho + 0.1\,r + 2.0, \quad . \quad . \quad . \quad . \quad . \quad (47)$$

in which t is the thickness of the abutment at the springing, ρ the radius, and r the rise,—all in feet. "The above formula applies equally to the smallest culvert or the largest bridge—whether circular or elliptical, and whatever the proportions of rise and span—and to any height of abutment. It applies also to all the usual methods of filling above the arch, whether with solid masonry to the level of the top of the crown, or entirely with earth. It gives a thickness of abutment which is safe in itself without any backing of earth behind it, and also safe against the pressure of the

* Concerning the method of determining the joint of rupture, see §§ 680–82.
† For a theoretical discussion of this subject, see §§ 711–12.

earth when the bridge is unloaded. It gives abutments which alone are safe when the bridge is loaded; but for small arches, the formula supposes that earth will be deposited behind the abutments to the height of the roadway. In small bridges and large culverts on first-class railroads, subject to the jarring of heavy trains at high speeds, the comparative cheapness with which an excess of strength can be thus given to important structures has led, in many cases, to the use of abutments from one fourth to one half thicker than those given by the preceding rule. If the abutment is of rough rubble, add 6 inches to the thickness by the above formula, to insure full thickness in every part." *

To find the thickness of the abutment at the bottom, lay off, in Fig. 139, $on = t$ as computed by the above equation; vertically above

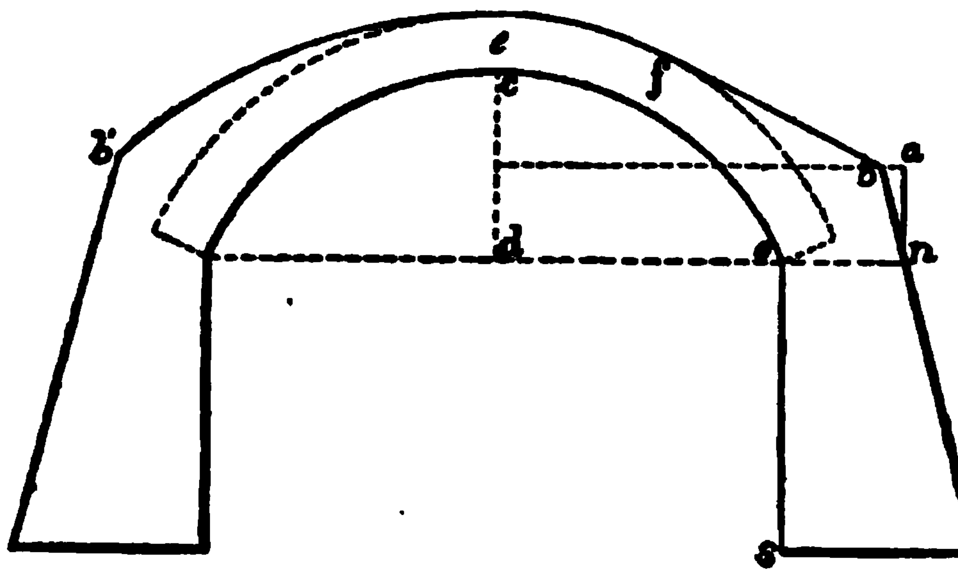

FIG. 139.

n lay off an = half the rise; and horizontally from a lay off ab = one twenty-fourth of the span. Then the line bn prolonged gives the back of the abutment, *provided* the width at the bottom, sp, is not less than two thirds of the height, os. "In practice, os will rarely exceed this limit, and only in arches of considerable rise. In very high abutments, the abutment as above will be too slight to sustain the earth pressure safely." *

To find the thickness of the arch, compute the thickness ce by equation (47), page 499, draw a curve through e parallel to the intrados, and from b draw a tangent to the extrados; and then will bfe be the top of the masonry filling above the arch. Or, instead of drawing the extrados as above, find, by trial, a circle which will pass through b, e, and b', the latter being a point on the left abutment corresponding to b on the right.

* Trautwine's Engineer's Pocket-book.

Trautwine's rule, or a similar one, for proportioning the abutment and the backing is frequently employed. For examples, see Plates IV and V.

723. *Rankine* says that in some of the best examples of bridges the thickness of the abutment ranges from *one third* to *one fifth* of the radius of curvature of the arch at its crown.

The following formula is said to represent *German* and *Russian practice*,

$$t = 1 + 0.04\,(5\,s + 4\,h), \quad . \quad . \quad . \quad . \quad . \qquad (48)$$

in which h is the distance between the springing line and the top of the foundation.

724. Dimensions of Actual Arches. Table 63 (pages 502–3) gives the dimensions of a number of actual structures, which, from their wide distribution and the frequency with which most of them are cited as examples, may be taken to represent average practice. Unfortunately the details concerning most of them are very meager, the following and those in the table being all that can be obtained.

No. 1 is the longest span ever built.

No. 2 is the longest span in existence.* The arch is a circular arc of 110°. It carries a conduit (clear diameter 9 feet) and a carriage-way (width 20 feet). The top of the roadway is 101 feet above the bottom of the ravine. The voussoirs are Quincy (Mass.) granite, and are 2 feet thick, 4 feet deep at the crown, and 6 feet at the springing. The spandrel filling is composed of Seneca sandstone, which, for a distance above the arch of 4 feet at the crown and 15 feet at the springing, is laid in regular courses with joints radial to the intrados; and hence the effective thickness of the arch is about 8 feet at the crown and about 21 feet at the springing (see Fig. 159, page 525). The abutments are prevented from spreading by the bed-rock in the side-hills.

No. 9 is a remarkable bridge. It was built by an "uneducated" mason in 1750; and although a very rude construction, is still in perfect condition. A former bridge of the same general design at the same place fell, on striking the centers, by the weight of the haunches forcing up the crown; and hence in building the present structure the load on the haunches of the arch was lightened by

* Concerning arched dams, see foot of page 330 and top of 331.

TABLE

DATA CONCERNING

Ref. No.	LOCATION AND DESCRIPTION.
1	Trezzo, Italy; built in 1380, destroyed in 1427; granite
2	Cabin John, Washington (D. C.); aqueduct; granite (see § 724, p. 501)
3	Grosvenor bridge, Chester, England
4	Ballochmyle, over the Ayr, Scotland
5	London bridge, England; street; granite
6	Gloucester, England
7	Turin, Italy
8	Alma bridge, Paris; small rough rubble in cement; railroad
9	Pont-y-Prydd, Wales; rough rubble in lime mortar (see § 724, p. 501)
10	Maidenhead, England; brick in cement; railroad
11	Neuilly, France; five spans (see page 504)
12	Bourbonnais Railway bridge; France; cut granite (see page 504)
13	Waterloo bridge, London, England; granite
14	Tongueland, England; turnpike
15	Napoleon bridge, Paris; small rough rubble in cement; railroad
16	Mantes, over Seine, France
17	Etherow river, England; railroad; four spans
18	Bishop Auckland, England; turnpike; built in 1388
19	Wellington bridge, Leeds, England
20	Louis XIX
21	Dean bridge, near Edinburgh, Scotland; turnpike
22	Licking Aqueduct, Chesapeake & Ohio canal
23	Dorlaston
24	Over the Oise, France; railroad
25	Triport, France; railroad
26	Conemaugh viaduct, Pennsylvania R. R.; sandstone in lime (no sand)
27	Royal Border viaduct, England; brick in cement
28	Posen viaduct, Germany; brick in cement
29	Orleans, France; railroad
30	Hutcheson bridge, Glasgow, Scotland
31	Falls bridge, Philadelphia & Reading R. R.
32	St. Maxence, over the Oise, France
33	Westminster bridge, London
34	Allentown, England; turnpike
35	Staines, England; turnpike
36	Black Rock Tunnel bridge, Philadelphia & Reading R. R.
37	Edinburgh
38	Swatara, Philadelphia & Reading R. R.; brick
39	Brent R. R. viaduct, England; brick in cement
40	Wellesley bridge at Limerick
41	Bow bridge, England; turnpike
42	Houghton river, England; railroad
43	Bewdly, England; turnpike
44	Chestnut Street bridge, Philadelphia; brick in cement
45	Carrollton viaduct, near Baltimore; railroad; granite
46	Llanwast, in Denbighshire, Wales; built in 1636; turnpike
47	Monocacy viaduct, Chesapeake & Ohio canal
48	Over the Forth, at Stirling
49	Nemours, France
50	Abattoir Street, Paris; railroad
51	Dôle, over the Doubs, France
52	Chateau Thierry, France
53	Avon viaduct, England; brick in cement
54	Filbert St., Extension Pennsylvania R. R., Philadelphia; brick in lime mortar
55	James River aqueduct, Virginia
56	Des Basses-Granges, Orléans à Tours, France
57	Over the Salat, France
58	Pesmes, over the Ougnon, France
59	Philadelphia & Reading R. R.
60	Couturette, Arbois, France
61	Tonoloway culvert, under Chesapeake & Ohio canal; rubble in cement

* C = semi-circle ; E = elliptical ; B = basket-handled.

63.

ACTUAL ARCHES.

Ref. No.	Engineer.	Curve of Intrados.	Span.	Rise.	Radius at Crown.	Thickness.	
						Crown.	Spring-ing.
	..	*	*feet.*	*feet.*	*feet.*	*feet.*	*feet.*
1		....	251	83	133	4.00	
2	Meigs................	C	220	57	134	4 †	6 †
3	Hartley..............	C	200	42	140	4.60	7.00
4	Miller.	C	181	90.5	90	4.50	6.00
5	Rennie...............	E	152	29.5	163	4.75	9.00
6	Telford.	E	150	35	98	4.50	
7	Mosca................	C	148	18	160	4.92	
8	Darcel...............	E	141	28	103	4.92	
9	Edwards..............	C	140	35	88	1.50	2.50
10	Brunel...............	E	128	24	169	5.25	7.16
11	Perronnet............	B	128	32	159	5.13	
12	Vaudray..............	C	124	6.92	281	2.67	3.60
13	Rennie...............	E	120	32	112	4.50	8.00
14	Telford..............	C	118	38	65	3.50	
15	Couche...............	C	116	14.8	120	4.00	
16		E	115	34		6.40	
17	Haskoll..............	C	100	25		4.00	4.00
18		C	100	22		1.88	1.83
19	Rennie...............	C	100	15		3.00	7.00
20	Perronnet....	C	94	9.75		3.67	
21	Telford..............	C	90	30	49	3.00	
22	Fisk.................	C	90	15	75	2.83	
23		C	87	13.5	76	3.50	
24		C	83	11.75		4.60	
25		E	81	28		4.45	
26		C	80	40	40	3.00	3.50
27		C	80	40	40	2.66	
28		C	80	16	58	4.66	
29		E	79	26.3		3.95	
30	Stephenson..........	C	79	18		3.50	4.50
31		C	78	25	43	3.00	
32	Perronnet............	C	77	6.40	119	4.80	
33	Labelye..............	C	76	38	38	7.60	14.00
34	Stephenson..........	C	75	11.50		2.50	3.00
35	Rennie...............	C	74	9.25		3.00	6.00
36	Robinson.............	C	72	16.5	47	2.75	2.75
37	Mylne................	C	72	36	36	2.75	
38	Osborne..............	C	70	25		3.50	3.50
39	Brunel...............	E	70	17.6	44	3.00	
40		E	70	17.5		2.00	
41	Walker...............	E	66	18.75	47	2.50	
42	Haskoll..............	C	65	32.5		2.75	2.75
43	Telford..............	C	60	20	33	2.20	
44	Kneass...............	C	60	18	34	2.50	
45		C	58	29	29	2.50	2.50
46	Jones................	C	58	17	33	1.50	
47	Fisk.................	E	54	9	45	2.50	
48		C	53	10.25		2.75	
49	Perronnet............	C	53	3.75		3.16	
50		C	53	5.11		2.97	
51		E	52	17.50		3.75	
52	Perronnet,...........	E	51	17.0		3.75	
53	Vignoles.............	E	50	15	28.3	2.00	
54		C	50	7		2.00	
55	Ellet	C	50	7	47	2.66	
56		C	49	24.5	24	3.95	
57		C	46	6.27		3.63	
58	Bertrand.............	C	45	3.83		3.83	
59	Steele...............	C	44	8	34	2.50	
60		C	43	6.13		2.97	
61	Fisk.................	C	40	15	21	2.00	

† See § 720, and also Fig. 159, page 525.

leaving horizontal cylindrical openings (see third paragraph of § 730) through the spandrel filling. The outer, or showing, arch stones are only 2.5 feet deep, and that depth is made up of two stones; and the inner arch stones are only 1.5 feet deep, and but from 6 to 9 inches thick. The stone quarried with tolerably fair natural beds, and received little or no dressing. It is a wagon-road bridge, and has almost no spandrel filling, the roadway being dangerously steep. A strain sheet of the arch shows that the line of resistance remains very near the center of the arch ring (see § 730). The mean pressure at the crown is about 244 pounds per square inch. On the whole it is an example of creditable engineering.

No. 11, as designed, had a radius at the crown of 160 feet; but the arch settled 2 feet on removing the center, and increased the radius to about 250 feet.

No. 12 is noted for its boldness. This design was tested by building an experimental arch—at Soupes, France—of the proportions given in the table, and 12 feet wide. The center of the experimental arch was struck after four months, when the total settlement was 1.25 inches, due mostly to the mortar joints, which were about one quarter inch; and it was not injured by a distributed load of 500 pounds per square foot, nor by a weight of 5 tons falling 1.5 feet on the key.

No. 46 is said to have "approached a horizontal line in consequence of the substitution of vehicles for pack-horses."

725. Table 63 affords some striking comparisons. For example, Nos. 8 and 9 have practically the same span; and as the rise of the former is four fifths that of the latter, the thickness at the crown of the former should be only about one and a quarter times that of the latter, while in fact it is 3.3 times as thick. However, the former carries a railroad, and the latter a turnpike; but, on the other hand, the former is laid in cement, and the latter in lime.

Nos. 11 and 12 have nearly the same span, but the rise of the former is 4.7 times the latter; and if the thickness at the crown were in like proportion—as it should be,—that of the former would be only 0.6 feet. Also compare No. 32 with No. 33; and No. 33 with Nos. 9 and 18.

726. Dimensions of Abutments. For examples of the abutments of railway culverts, see Tables 49–52 (pages 425–31). Table 64,

below, gives the dimensions of a number of abutments representative of French railroad practice.

TABLE 64.

DIMENSIONS OF ABUTMENTS FROM FRENCH RAILROAD PRACTICE.*

Ref. No.	DESIGNATION OF BRIDGE.	Span.	Rise.	Depth of Keystone.	Height of Abutment.	Mean thickness of Abutment.
	CIRCULAR ARCHES.	feet.	feet.	feet.	feet.	feet.
1	De crochet, chemin de fer de Paris à Chartres	13.2	....	1.65	13.20	4.95
2	De Long-Sauts, chemin de fer de Paris à Chartres.	16.5	....	1.81	9.90	5.90
3	D'Enghien, chemin de fer du Nord	24.4	....	1.95	6.60	6.93
4	De Pantin, canal St. Martin	27.0	....	2.47	11.85	10.55
5	De la Bastille, canal St. Martin	36.3	....	3.95	20.75	9.90
6	De Basses-Granges, Orleans à Tours	49.4	....	3.95	6.60	12.50
	SEGMENTAL ARCHES.					
7	Des Fruitiers, chemin du fer du Nord	13.2	2.31	1.81	13.20	5.94
8	De Paisia	16.5	2.64	1.72	6.60	5.61
9	De Méry, chemin de fer du Nord	25.2	2.97	2.14	14.20	11.71
10	De Couturette, at Arbois	42.9	6.13	2.97	6.60	17.16
11	Over the Salat	46.1	6.27	3.63	24.49	19.14
12	De la rue des Abattoirs, at Paris, chemin de fer de Strasbourg	52.9	5.11	2.97	12.96	33.00
13	Over the Forth, at Stirling	53.5	10.25	2.75	20.75	16.00
14	St. Maxence, over the Oise	77.2	6.40	4.80	27.85	33.94
15	Over the Oise, chemin de fer du Nord	82.7	11.75	4.60	17.90	31.65
16	De Dorlaston	87.0	13.50	3.50	16.55	32.20
	ELLIPTICAL OR FALSE-ELLIPTICAL ARCHES.					
17	De Charolles	19.8	7.55	1.95	1.30	5.25
18	Du Canal St. Denis	39.5	14.85	2.95	10.20	12.35
19	De Chateau-Thierry	51.3	17.10	3.75	13.65	15.00
20	De Dôle, over the Doubs	52.4	17.50	3.75	1.85	11.85
21	Wellesley, at Limerick	70.0	17.50	2.00	12.00	16.50
22	D'Orleans, chemin de fer de Vierzon	79.5	26.30	3.95	2.85	18.40
23	De Trilport	80.7	27.80	4.45	6.40	19.30
24	De Nantes, over the Seine	115.2	34.40	6.40	8.20	28.90
25	De Neuilly, over the Seine	128.0	32.00	5.35	7.55	35.50

727. ILLUSTRATIONS OF ACTUAL ARCHES.—For illustrations of stone arches for railroad culverts, see Plates II–V. Fig. 143 (page 509) shows a 50-foot stone arch on the Pennsylvania Railroad. For brick arches for sewers, see Figs. 148 and 149 (pages 513 and 514). For an example of a brick tunnel-arch, see Fig. 147 (page 512). Cabin John arch, the longest span in the world (see No. 2 of Table 63, page 502), is shown incidentally in Fig. 159 (page 525).

728. MINOR DETAILS. Backing. The backing is masonry of inferior quality laid outside and above the arch stones proper, to give additional security. The backing is ordinarily coursed or random rubble, but sometimes concrete. Sometimes the upper ends

* E. Sherman Gould, in Van Nostrand's Engin'g Mag., vol. xxix, p. 450.

of the arch stones are cut with horizontal surfaces, in which case the backing is built in courses of the same depths as these steps and bonded with them. The backing is occasionally built in radiating courses, whose beds are prolongations of the bed-joints of the arch stones; but it usually consists of rubble, laid in horizontal courses abutting against the arch ring, with occasional arch stones extending into the former to bond both together. The radial joints possess some advantages in stability and strength, particularly above the joint of rupture; but below that joint the horizontal and vertical joints are best, since this form of construction the better resists the overturning of the arch outward about the springing line. Ordinarily, the backing has a zero thickness at or near the crown, and gradually increases to the springing line; but sometimes it has a considerable thickness at the crown, and is proportionally thicker at the springing.

It is impossible to compute the degree of stability obtained by the use of backing; but it is certain that the amount ordinarily employed adds very greatly to the stability of the arch ring. In fact, many arches are little more than abutting cantilevers; and it is probable that often the backing alone would support the structure, if the arch ring were entirely removed.

729. Spandrel Filling. Since the roadway must not deviate greatly from a horizontal line, a considerable quantity of material is required above the backing to bring the roadway level. Ordinarily this space is filled with earth, gravel, broken stone, cinders, etc. Sometimes, to save filling, small arches are built over the haunches of the main arch, as shown in Fig. 140. The interior longitudinal walls may be strengthened by transverse walls between them. To distribute the pressure uniformly, the feet of these walls should be expanded by footings where they rest upon the back of the arch.

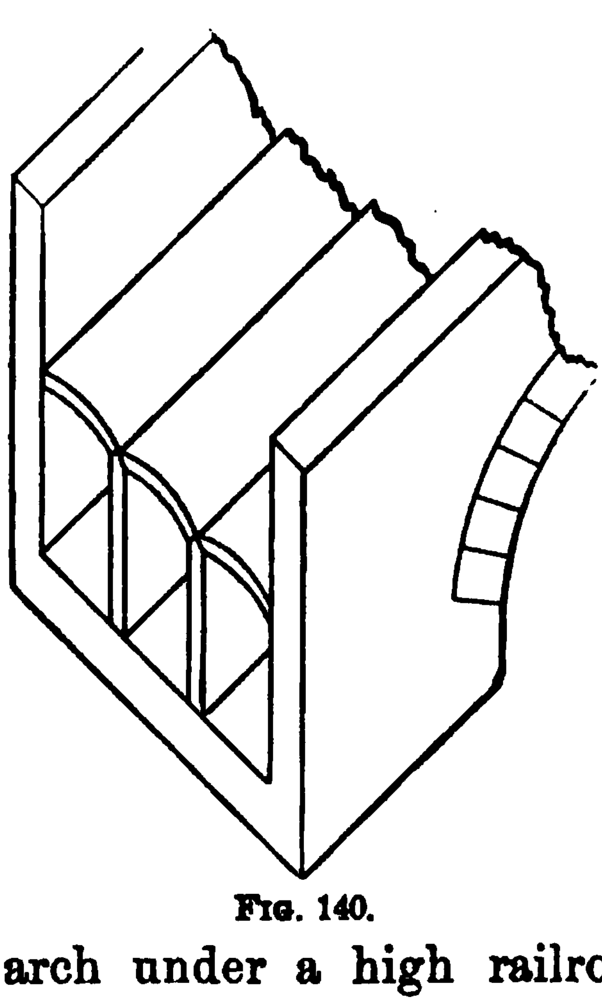

FIG. 140.

730. When the load is entirely stationary—as in an aqueduct or canal bridge—or nearly so—as in a long span arch under a high railroad embankment,—the materials of the

spandrel filling and the size and position of the empty spaces may be such as to cause the line of resistance to coincide, at least very nearly, with the center of the arch ring.

For example, *ABCD*, Fig. 141, represents a semi-arch for which it is required to find a disposition of the load that will cause the line of resistance to coincide with the center line of the arch ring.

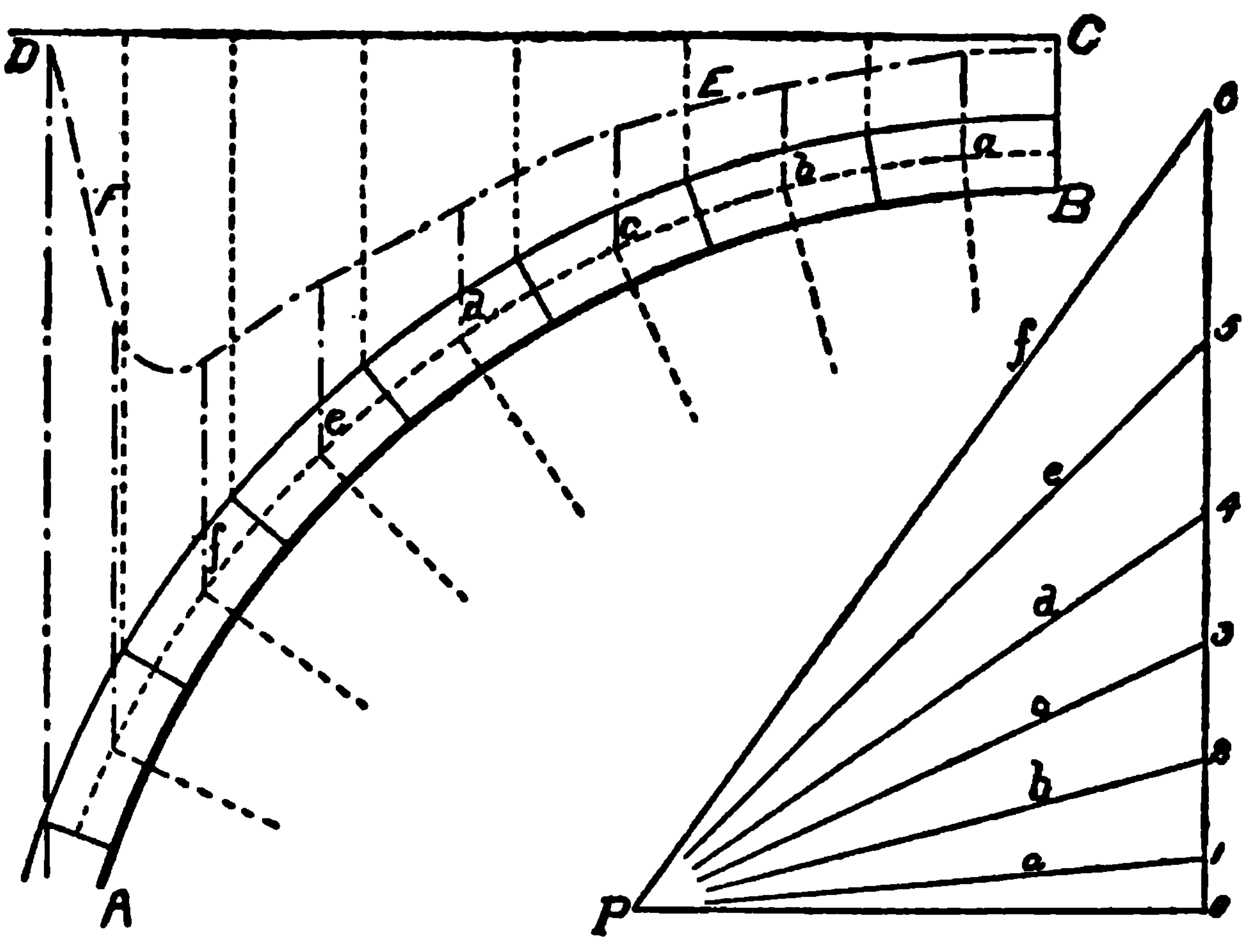

FIG. 141.

Divide the arch and the load into any convenient number of divisions, by vertical lines as shown. From *P* draw radiating lines parallel to the tangents of the center line of the arch ring at *a, b, c*, etc.,—the middle points of the successive divisions;—and then at such a distance from *P* that 01 shall represent, to any convenient scale, the load on the first section of the arch ring (including its own weight), draw a vertical line through 0. The intercepts 0–1, 1–2, 2–3, etc., represent, to scale, the loads which the several divisions must have to cause the line of resistance to coincide with the center of the arch ring. Lay off the distances 0-1, 1–2, etc., at the centres of the respective sections vertically upwards from the center line of the arch ring, and trace a curve through their upper ends. The line thus formed—*EF*, Fig. 141—shows the required amount of homogeneous load; *i. e.*, *EF* is the contour of the homogeneous load that will cause the line of resistance to pass through the center of each joint.

Hence, by choosing the material of the spandrel filling and

arranging the empty spaces so that the actual load shall be equivalent in intensity and distribution to the reduced load obtained as above, the voussoirs can be made of moderate depth. The vacant spaces may be obtained by the method shown in Fig. 140 (page 506) ; or by that shown in Fig. 142, in which A is a small empty cylindrical arch extending from the face of one end wall to that of the other. (See the description of arch No. 9, § 724, p. 501.)

Notice that the lines radiating successively from P, Fig. 141 (page 507), will intercept increasing lengths on the load-line ; and that, therefore, the load which will keep a circular arch in equilibrium must increase in intensity per horizontal foot, from the crown towards the springing, and must become infinite at the springing of a semi-circular arch. Hence it follows that it is not practicable to load a circular arch, beyond a certain distance from the crown, so that the line of resistance shall coincide with the center line of the arch ring.

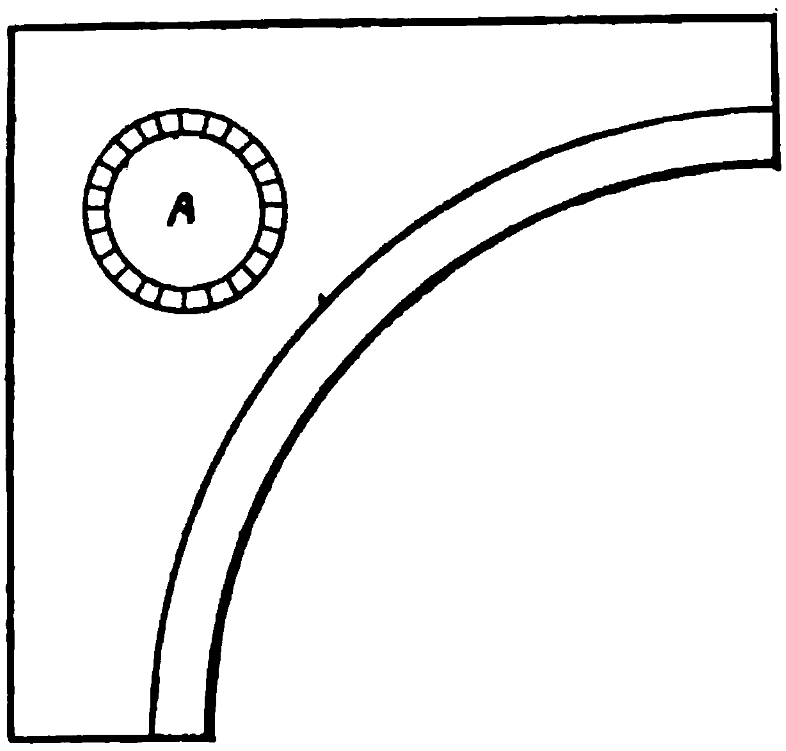

Fig. 142.

731. Drainage. The drainage of arch bridges of more than one span is generally effected by giving the top surface of the backing a slight inclination from each side toward the center of the width of the bridge and also from the center toward the end of the span. The water is thus collected over the piers, from whence it is discharged through pipes laid in the masonry.

To prevent leakage through the backing and through the arch sheeting, the top of the former should be covered with a layer of puddle, or plastered with a coat of best cement mortar (see § 141), or painted with coal tar or asphaltum (see § 264).

732. For an illustration of the method of draining a series of arches, and also of several minor details not mentioned above, see Fig. 143, which represents "Little Juniata bridge No. 12" on the Pennsylvania Railroad.*

* Published by permission of Wm. H. Brown, chief engineer.

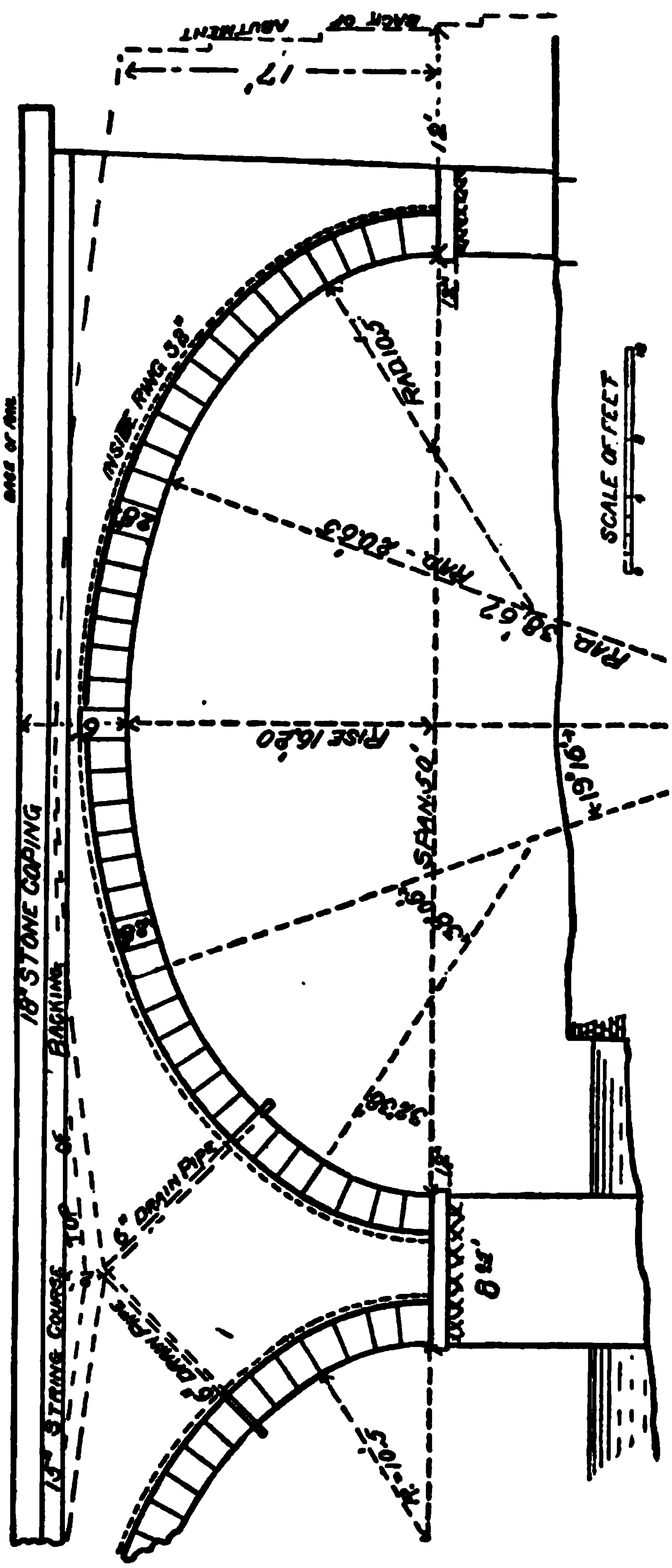

FIG. 148.—Little Juniata Bridge No. 12, Pennsylvania R. R.

733. BRICK ARCHES. The only matter requiring special mention in connection with brick arches is the bond to be employed. When the thickness of the arch exceeds a brick and a half, the bond from the soffit outward is a very important matter. There are three principal methods employed in bonding brick arches. (1) The arch may be built in concentric rings; *i. e.*, all the brick may be laid as stretchers, with only the tenacity of the mortar to unite the several rings (see Fig. 144). This form of construction is frequently called *rowlock bond.* (2) Part of the brick may be laid as stretchers and part as headers, as in ordinary walls, by thickening the outer ends of the joints—either by using more mortar or by driving in thin pieces of slate,—so that there shall be the same number of bricks in

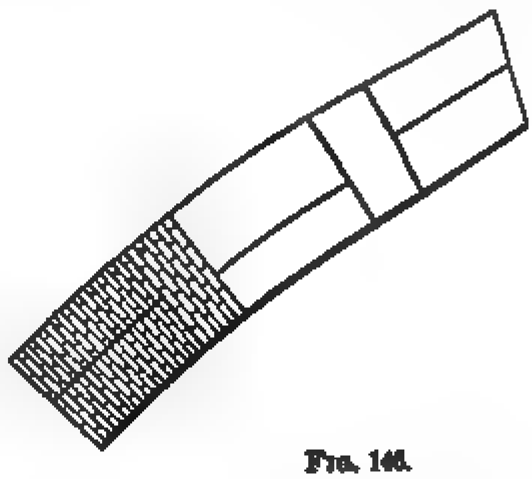

FIG. 144. FIG. 145. FIG. 146.

each ring (see Fig. 145). This form of construction is known as *header and stretcher bond,* or is described as being laid with *continuous radial joints.* (3) *Block in course bond* is formed by dividing the arch into sections similar in shape to the voussoirs of stone arches, and laying the brick in each section with any desired bond, but making the radial joints between the sections continuous from intrados to extrados. With this form of construction, it is customary to lay one section in rowlock bond and the other with radial joints continuous from intrados to extrados, the latter section being much narrower than the former (see Fig. 146).

1. The objection to laying the arch in concentric rings is that, since the rings act nearly or quite independent of each other, the proportion of the load carried by each can not be determined. A ring may be called upon to support considerably more than its proper share of the load. This is by far the most common form of bonding in brick arches, and that this difficulty does not more often mani-

fest itself is doubtless due to the very low unit working pressure employed. The *mean* pressure on brick masonry arches ordinarily varies from 20 to 40 pounds per square inch, under which condition a single ring might carry the entire pressure (see Tables 19 and 20, pages 164 and 166). The objection to this form of bond can be partially removed by using the very best cement mortar between the rings.

The advantages of the ring bond, particularly for tunnel and sewer arches, are as follows: It gives 4-inch toothings for connecting with the succeeding section, while the others give only 2-inch toothings along much of the outline. It requires less cement, is more rapidly laid, and is less liable to be poorly executed. It possesses certain advantages in facilities for drainage, when laid in the presence of water.

2. The objection to laying the arch with continuous radial joints is that the outer ends of the joints, being thicker than the inner, will yield more than the latter as the centers are removed, and hence concentrate the pressure on the intrados. This objection is not serious when this bond is employed in a narrow section between two larger sections laid in rowlock courses (see Fig. 146).

3. When the brickwork is to be subject to a heavy pressure, some form of the block in course bond should be employed. For economy of labor, the "blocks" of headers should be placed at such a distance apart that between each pair of them there shall be one more course of stretchers in the outer than in the inner ring; but a moment's consideration will show that this would make each section about half as long as the radius of the arch,—which, of course, is too long to be of any material benefit. Hence, this method necessitates the use of thin bricks at the ends of the rings.

734. Examples of Brick Arches. The method of bonding shown in Fig. 146 (page 510) is frequently employed—as, for example, in the 70-foot brick arch of the Swatara bridge (Philadelphia and Reading R. R.). The bonding employed in arching the Vosburg tunnel (Lehigh Valley R. R.) is shown in Fig. 147 (page 512).*

735. Fig. 148 (page 513) shows the standard forms of large brick sewers employed in the city of Philadelphia.† "They are

* From Rosenberg's "The Vosburg Tunnel," by permission.

† R. Hering, in Trans. Am. Soc. of C. E., vol. vii, pp. 252–57. The illustrations are reproduced from those in the original, the force diagrams being omitted here.

designed for a maximum pressure on the brick-work of 80 pounds per square inch," which, considering the usual specifications for such work (see § 260, p. 176), seems unnecessarily small (see Tables 19 and 20, pages 164 and 166).

Fig. 149 (page 514) shows the standard forms of sewers in Washington, D. C.* "The invert as shown is the theoretical form, although the concrete is rammed into the trench and nearly always extends beyond the limits shown." The largest sewers have a trap-rock bottom; the intermediate sizes have a semi-circular vitrified

FIG. 147.—Bond and Center of Vosburg Tunnel.

pipe in the bottom; and the smallest sizes consist of sewer pipes bedded in concrete.

736. Owing to their great number of joints, brick arches are liable to settle much more than stone ones, when the centers are removed; and hence are less suitable than the former for large or flat arches. Nevertheless a number of brick arches of large span have been built (see Table 63, page 502). Trautwine gives the following description of some bold examples. "On the Filbert Street extension of the Pennsylvania R. R., in Philadelphia, are four brick arches of 50 feet span, and with the very low rise of 7 feet. The arch rings are $2\frac{1}{2}$ feet thick, except on their showing faces, where they are but 2 feet. The joints are in common lime mortar, and are about $\frac{1}{4}$ inch

* Report of the Commissioners of the District of Columbia, for the year ending June 30, 1884, p. 175. For details of quantities of material required, and for estimates of cost, see report for preceding year, pp. 277–79.

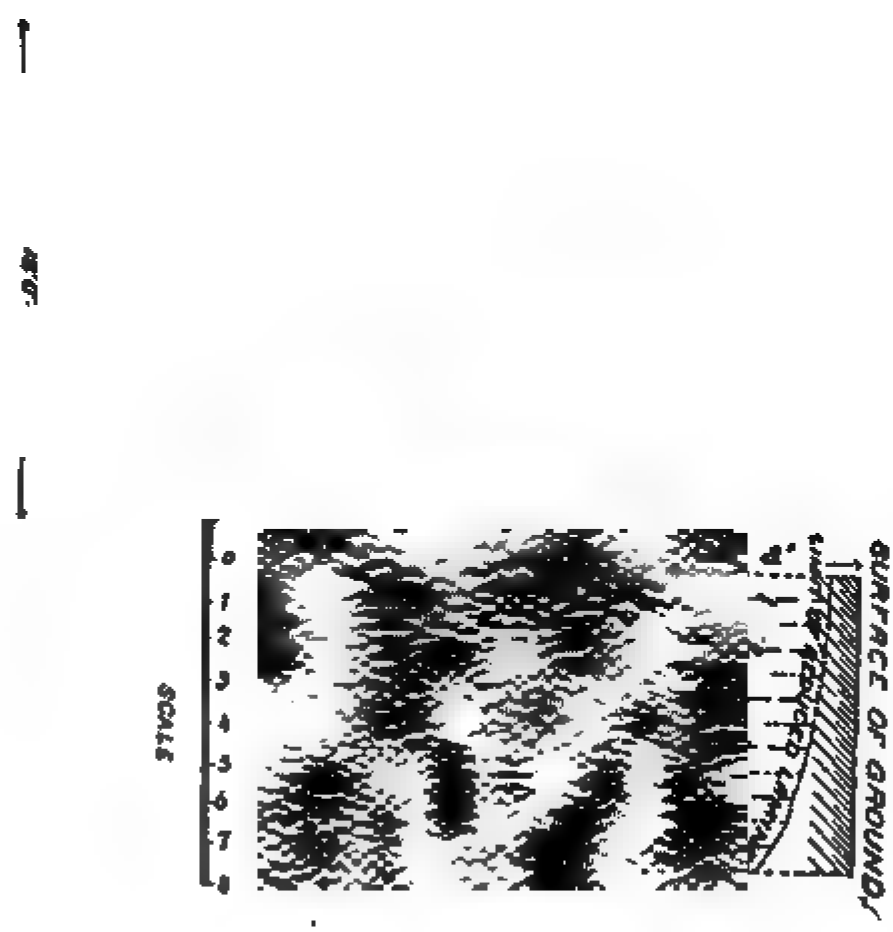

FIG. 148.—Brick Arches for Large Sewers.—Philadelphia Standard.

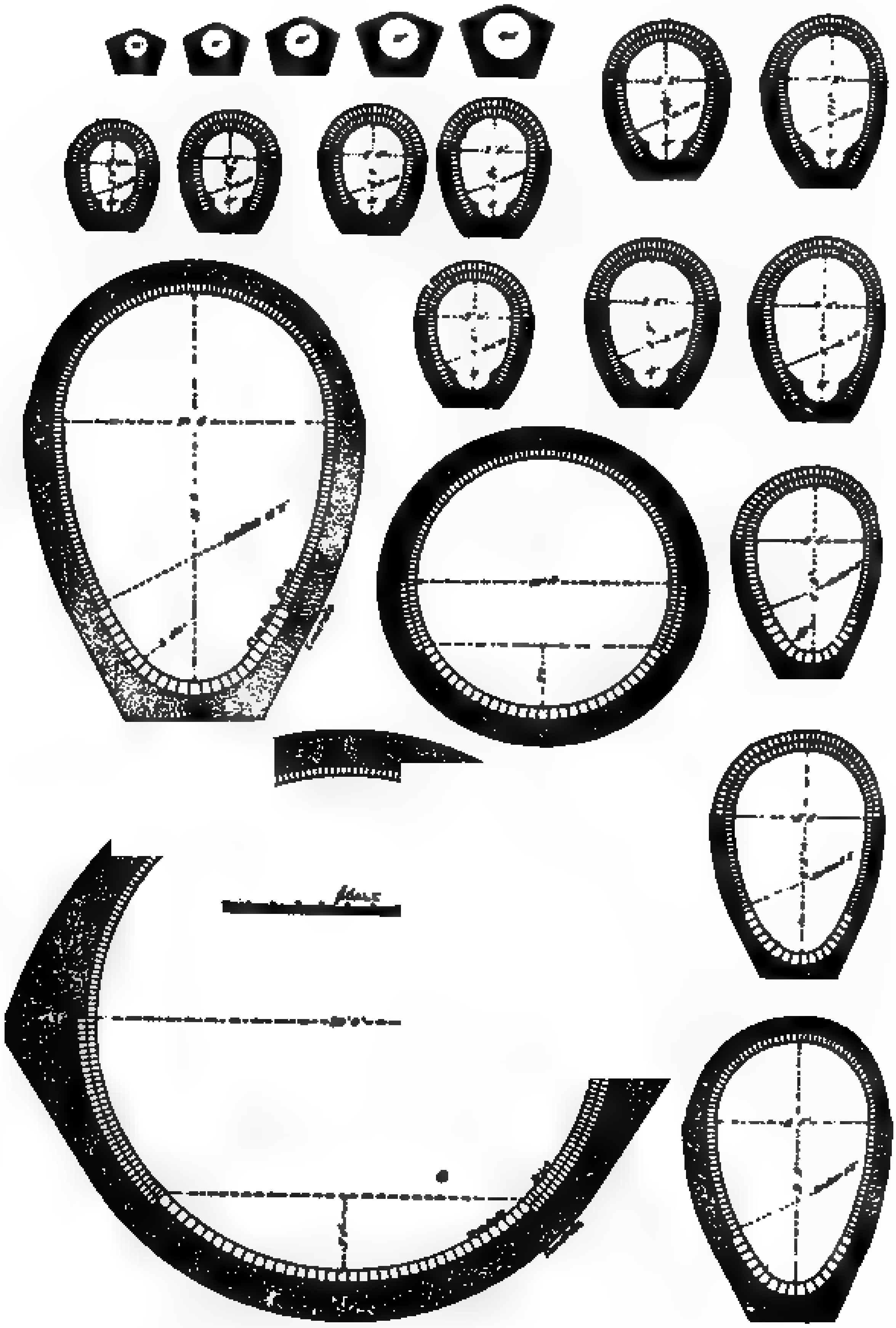

FIG. 149.—Standard Forms of Brick Sewers.—Washington, D. C.

thick. These four arches, about 200 yards apart, with a large number of others of 26 feet span, form a viaduct. The piers between the short spans are 4½ feet thick, and those at the ends of the 50-foot spans are 18½ feet. The road-bed is about 100 feet wide, giving room for 9 or 10 tracks. The springing lines of all the arches are about 6 to 8 feet above the ground. One of the 50-foot arches settled 3 inches upon permanently striking the center; but no further settlement has been observed, although the viaduct has, since built (1880), had a very heavy freight and passenger traffic at from 10 to 20 miles per hour."

737. SPECIFICATIONS FOR STONE ARCHES. The specifications for arch masonry employed on the Atchison, Topeka and Santa Fé Railroad are as follows: *

738. First-Class Arch Masonry shall be built in accordance with the specifications for first-class masonry [see § 207], with the exception of the arch sheeting and ring stones. The ring stones shall be dressed to such shape as the engineer shall determine. The ring stones and the arch sheeting shall be not less than ten inches (10") thick on the intrados, and shall have a depth equal to the specified thickness of the arch. The joints shall be at right angles to the intrados, and their thickness shall not exceed three eighths of an inch (⅜"). The face of the sheeting stones shall be dressed so as to make a close centering joint. The ring stones and sheeting shall break joints not less than one foot (1').

The wings shall be neatly stepped with selected stones of the full width of the wing, and of not less than ten inches (10") in thickness, overlapping by not less than one and one half feet (1½'); or they shall be finished with a neatly capped newel at the end of each wing, and a coping course on the wing. The parapets shall be finished with a coping course of not less than ten inches (10") in thickness, having a projection of six inches (6").

739. Second-Class Arch Masonry shall be the same as first-class masonry (see § 207). The stones of the arch sheeting shall be at least four inches (4") in thickness on the intrados; shall have a depth equal to the thickness of the arch; shall have good bearings throughout; and shall be well bonded to each other and to the ring stones.

740. SPECIFICATIONS FOR BRICK ARCHES. See §§ 260–61 (pages 176–77).

ART. 3. ARCH CENTERS.

741. A *center* is a temporary structure for supporting an arch while in process of construction. It usually consists of a number of frames (commonly called *ribs*) placed a few feet apart in planes

* For general specifications for railroad masonry, see Appendix I.

perpendicular to the axis of the arch, and covered with narrow planks (called *laggings*) running parallel to the axis of the arch, upon which the arch stones rest. The center is usually wood—either a solid rib or a truss,—but is sometimes a curved rolled-iron beam. In a trussed center, the pieces upon which the laggings rest are called *back-pieces*. The ends of the ribs may be supported by timber struts which abut against large timbers laid upon the ground, or they may rest upon a shoulder on the abutment.

The framing, setting up, and striking of the centers (§§ 752–55) is a very important part of the construction of any arch, particularly one of long span. A change in the shape of the center, due to insufficient strength or improper bracing, will be followed by a change in the curve of the intrados and consequently of the line of resistance, which may endanger the safety of the arch itself.

742. Load to be Supported. If there were no friction, the load to be supported by the center could be computed exactly; but friction between the several arch stones and between these and the center renders all formulas for that purpose very uncertain. Fortunately, the exact load upon the center is not required; for the center is only a temporary structure, and the material employed in its construction is not entirely lost. Hence it is wise to assume the loads to be greater than they really will be. Some allowance must also be made for the accumulation of the material on the center and for the effect of jarring during erection. The following analysis of the problem will show roughly what the forces are and why great accuracy is not possible.

To determine the pressure on the center, consider the voussoir $DEFG$, Fig. 150, and let

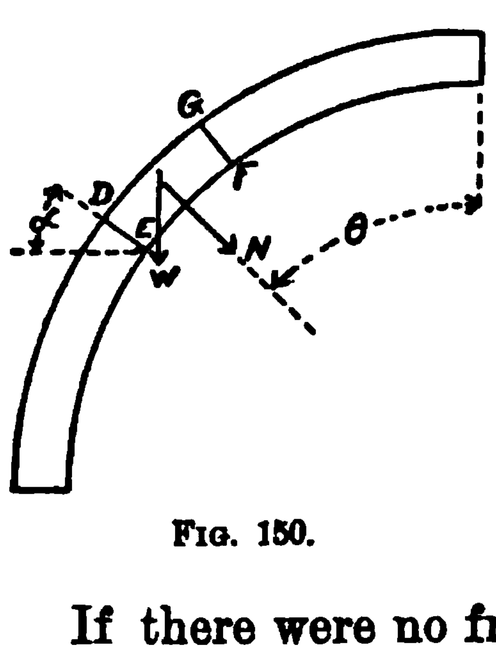

Fig. 150.

α = the angle which the joint DE makes with the horizontal;

μ = the co-efficient of friction (see Table 36, page 315), *i. e.*, μ is the tangent of the angle of repose;

θ = the angular distance of any point from the crown;

W = the weight of the voussoir $DEFG$;

N = the radial pressure on the center due to the weight of $DEFG$.

If there were no friction, the stone $DEFG$ would be supported

by the normal resistance of the surface DE and the radial reaction of the center. The pressure on the surface DE would then be $W \cos \alpha$, and the pressure in the direction of the radius $W \sin \alpha$.

Friction causes a slight indetermination, since part of the weight of the voussoirs may pass to the abutment either through the arch ring or through the back-pieces (perimeter) of the center. Owing to friction, both of these surfaces will offer, in addition to the above, a resistance equal to the product of the perpendicular pressure and the co-efficient of friction (foot-note, page 276). If the normal pressure on the joint DE is $W \cos \alpha$, then the frictional resistance is $\mu W \cos \alpha$. Any frictional resistance in the joint DE will decrease the pressure on the center by that amount; and consequently, with friction on the joint DE, the radial pressure on the center is

$$N = W(\sin \alpha - \mu \cos \alpha). \qquad (49)$$

On the other hand, if there is friction between the arch stone and the center, the frictional resistance between these surfaces will decrease the pressure upon the joints DE, as computed above; and consequently the value of N will not be as in equation (49).

Notice that in passing from the springing toward the crown the pressure of one arch stone on the other decreases. Near the crown this decrease is rapid, and consequently the friction between the voussoirs may be neglected. Under this condition, the radial pressure on the center is

$$N = W \cos \theta. \qquad (50)$$

As a rough approximation, equation (50) may be applied for the first 30° from the crown, although it gives results slightly greater than the real pressures; and for the second 30°, equation (49) may be employed, although it gives results less than the actual pressure; and for the third 30°, the arch stones may be considered self-supporting.

743. The value of the co-efficient to be employed in equation (49) is somewhat uncertain. Disregarding the adhesion of the mortar, the co-efficient varies from about 0.4 to 0.8 (see Table 36,

page 315); and, including the adhesion of good cement mortar, it may be nearly, or even more than, 1. (It is 1 if an arch stone remains at rest, without other support, when placed upon another one in such a position that the joint between them makes an angle of 45° with the horizontal.) If the arch is small, and consequently laid up before the mortar has time to harden, probably the smaller value of the co-efficient should be used; but if the arch is laid up so slowly that the mortar has time to harden, a larger value could, with equal safety, be employed. As a general average, we will assume that the co-efficient is .58, *i. e.*, that the angle of repose is 30°.

Notice that by equation (49) $N = 0$, if $\tan\ \alpha = \mu$; that is to say, $N = 0$, if $\alpha = 30°$. This shows that as the arch stones are placed upon one another they would not begin to press upon the center rib until the plane of the lower face of the top one reaches an angle of 30° with the horizon.

Table 65 gives the value of the radial pressure of the several portions of the arch upon the center; and also shows the difference between applying equation (49) and equation (50). Undoubtedly the former should be applied when the angle of the lower face of any arch stone with the horizontal does not differ greatly from 30°; and when this angle is nearly 90°, then equation (50) should be applied. It is impossible to determine the point at which one equation becomes inapplicable and the other applicable; but it is probably safe to apply equation (49) up to 60° from the horizontal.

744. *Example.* To illustrate the method of using Table 65, assume that it is required to find the pressure on a back-piece of a 20-foot semi-circular arch which extends from 30° to 60° from the horizontal, the ribs being 5 feet apart, and the arch stones being 2 feet deep and weighing 150 pounds per cubic foot. Take the sum of the decimals in the middle column of Table 65, which is 3.19. This must be multiplied by the weight of the arch resting on 2° of the center. (In this connection it is convenient to remember that an arc of 1° is equal to 0.0175 times the radius.) The radius to the middle of the voussoir is 11 feet, and the length of 2° of arc is 0.38 feet. The volume of 2° is $0.38 \times 5 \times 2 = 3.8$ cubic feet; and the weight of 2° is $3.8 \times 150 = 570$ pounds. Therefore the pressure on the back-piece is $570 \times 3.19 = 1{,}818$ pounds.

TABLE 65.

THE RADIAL PRESSURE OF THE ARCH STONES OF A SEMI-ARCH, ON THE CENTER.

Angle of the Lower Face with the Horizontal.	Radial Pressure in Terms of the Weight of the Arch Stone.	
	By Equation (49).	By Equation (50).
30°	0.00	
32°	0.04	
34°	0.08	
36°	0.12	
38°	0.16	
40°	0.20	
42°	0.24	0.67
44°	0.28	0.69
46°	0.32	0.72
48°	0 36	0.74
50°	0.40	0.76
55°	0.45	0.82
60°	0.54	0.86
65°	....	0.91
70°	....	0.94
80°	....	0.98
90°	....	1.00

745. OUTLINE FORMS OF CENTERS. Solid Wooden Rib. For flat arches of 10-foot span or under, the rib may consist of a plank, *a, a*, Fig. 151, 10 or 12 inches wide and 1½ or 2 inches thick, set

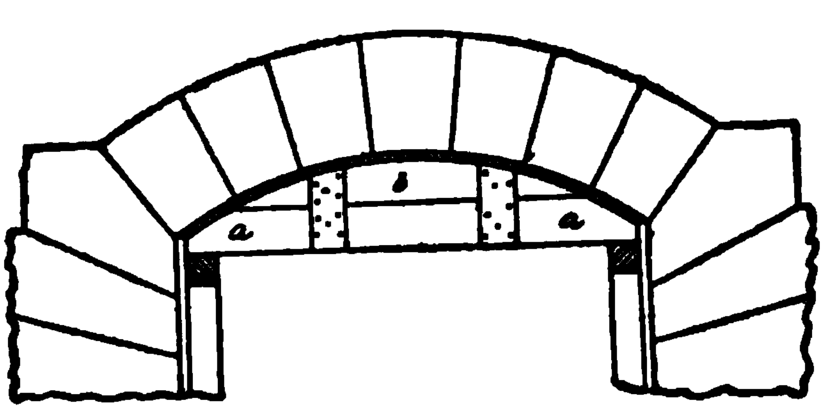

FIG. 151.

edgewise, and another, *b*, of the same thickness, trimmed to the curve of the intrados and placed above the first. The two should be fastened together by nailing on two cleats of narrow boards as shown. These centers may be placed 2 or 3 feet apart.

746. Built Wooden Rib. For flat arches of 10 to 30 feet span, the rib may consist of two or three thicknesses of short boards, fitted and nailed (or bolted) together as shown in Fig. 152. An iron plate is often bolted on over the joints (see Fig. 147, page 512), which adds materially to the rigidity of the rib. Centers of this form have an astonishing strength. Trautwine gives the two following examples which strikingly illustrate this.

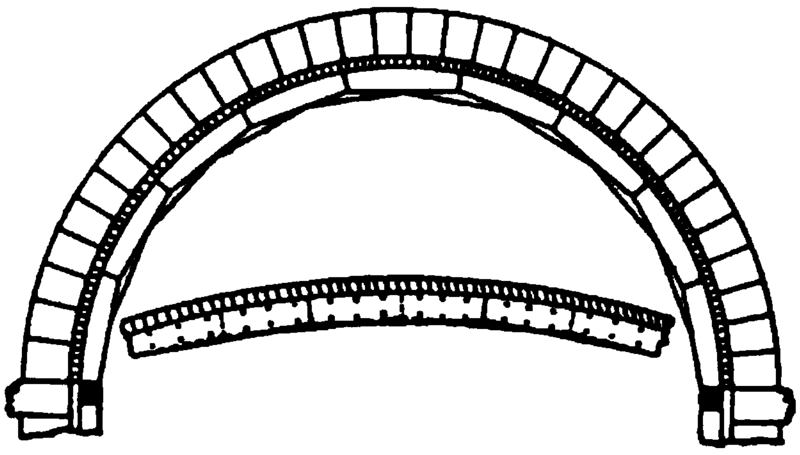

FIG. 152.

In the first of these examples, this form of center was employed for a semi-circular arch of 35 feet span, having arch stones 2 feet deep. "Each rib consisted of two thicknesses of 2-inch plank, in lengths of about 6.5 feet, treenailed together so as to break joint. Each piece of plank was 12 inches deep at the middle, and 8 inches at each end, the top edge being cut to suit the curve of the arch. The treenails were 1.25 inches in diameter, and 12 of them were used to each length. These ribs were placed 17 inches apart from center to center, and were steadied together by a bridging piece of 1-inch board, 13 inches long, at each joint of the planks, or about 3.25 feet apart. Headway for traffic being necessary under the arch, there were no chords to unite the opposite feet of the ribs. The ribs were covered with close board-lagging, which also assisted in steadying them together transversely. As the arch approached about two thirds of its height on each side, the ribs began to sink at the haunches and rise at the crown. This was rectified by loading the crown with stone to be used in completing the arch, which was then finished without further trouble."

The other example was an elliptic arch of 60 feet span and 15 feet rise, the arch stones being 3 feet deep at the crown and 4 feet at the springing. "Each frame of the centre was a simple rib 6 inches thick, composed of three thicknesses of 2-inch oak plank, in lengths (about 7 to 15 feet) to suit the curve and at the same time to preserve a width of about 16 inches at the middle of each length and 12 inches at each of its ends. The segments broke joints, and were well treenailed together with from ten to sixteen

treenails to each length. There were no chords. These ribs were placed 18 inches from center to center, and were steadied against one another by a board bridging-piece, 1 foot long, at every 5 feet. When the arch stones had approached to within about 12 feet of each other, near the middle of the span, the sinking at the crown and the rising at the haunches had become so alarming that pieces of 12- × 12-inch oak were hastily inserted at intervals and well wedged against the arch stones at their ends. The arch was then finished in sections between these timbers, which were removed one by one as the arch was completed."

Although the above examples can not be commended as good construction—the flexibility of the ribs being so great as to endanger the stability of the arch during erection and to break the adhesion of the mortar, thus decreasing the strength of the finished arch,—they are very instructive as showing the strength attainable by this method.

747. The above form of center is frequently employed, particularly in tunnels, for spans of 20 to 30 feet, precautions being taken to have the pieces break joints, to secure good bearings at the joints, and to nail or bolt the several segments firmly together. The centers for the 25-foot arch of the Musconetcong (N. J.) tunnel (Lehigh Valley R. R.) consisted of segments of 3-inch plank, 5 feet 8 inches long, 14 inches wide at the center, and 8 inches at the ends, bolted together with four $\frac{1}{2}$-inch and four $\frac{3}{4}$-inch bolts each, and 14- × 8-inch pieces of plate-iron over the joints. Where the center was required to support the earth also, a three-ply rib was employed; but in other positions two-ply ribs, spaced 4 to 5 feet apart, were used. Centers of this form have successfully stood very bad ground in the Musconetcong and other tunnels;* and hence we may infer that they are at least sufficiently strong for any ordinary arch of 30 feet span.

Although not necessary for stability, it is wise to connect the feet of the rib by nailing a narrow board on each side, to prevent the end of the rib from spreading outwards and pressing against the masonry—thus interfering with the striking of the center,—and also to prevent deformation in handling it.

* Drinker's Tunneling, p. 548.

748. Braced Wooden Rib. For semi-circular arches of 15 to 30 feet span, a construction similar to that shown in Fig. 147 (page 512) may be employed. The segments should consist of two thicknesses of 1- or 2-inch plank, according to span, from 8 to 12 inches wide at the middle, according to the length of the segments. The horizontal chord and the vertical tie may each be made of two thicknesses of the plank from which the segments are made.

For greater rigidity, the rib may be further braced by any of the methods shown in outline in Figs. 153, 154, 155, or by obvious

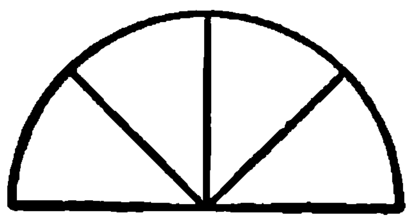

FIG. 153.

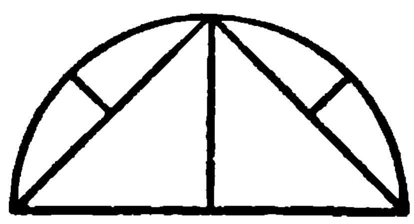

FIG. 154.

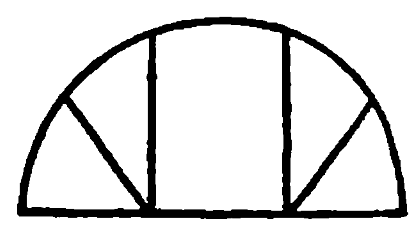

FIG. 155.

modifications of them. The form to be adopted often depends upon the passage-way required under the arch. Fig. 153 is supported by a post under each end; in extreme cases, Fig. 154 may be supported at the middle point also; and Fig. 155 may be supported at both middle points as well as at the ends.

Since the arch masonry near the springing does not press upon the center, it may be laid with a template before the center is set up; and hence frequently the center of a semi-circular arch does not extend down to the springing line. For examples, see Figs. 147 and 158 (page 512 and 524).

Center frames are put together on a temporary platform or the floor of a large room, on which a full-size drawing of the rib is first drawn.

749. Trussed Center. When the span is too great to employ any of the centers described above, it becomes necessary to construct trussed centers. It is not necessary here to discuss the principles

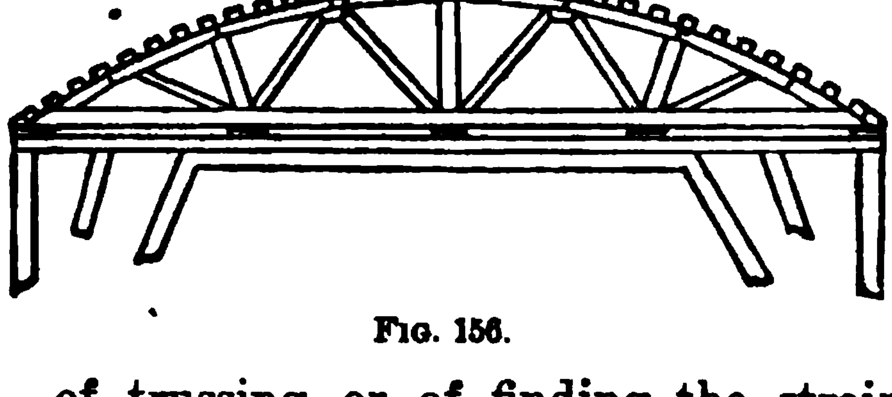

FIG. 156.

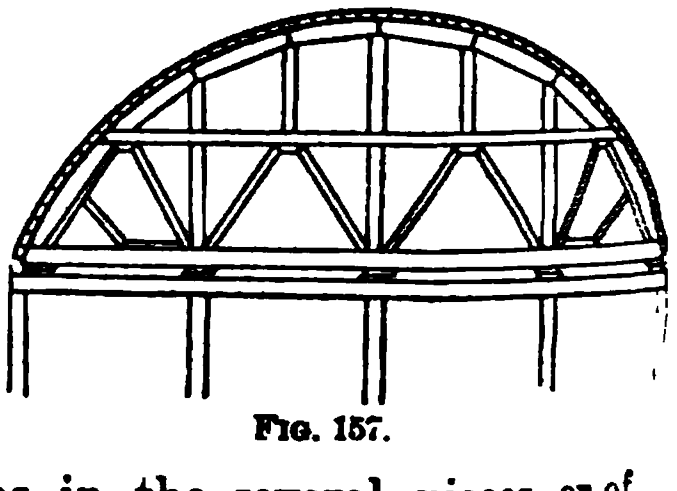

FIG. 157.

of trussing, or of finding the strains in the several pieces, or of determining the sections, or of joining the several pieces,—all of

which are fully described in treatises on roof and bridge construction. There is a very great variety of methods of constructing such centers. Figs. 156 and 157 show two common, simple, and efficient general forms.

750. CAMBER. Strictly, the center should be constructed with a camber just equal to the amount it will yield when loaded with the arch; but, since the load is indeterminate, it is impossible to compute what this will be. Of course, the camber depends upon the unit strain in the material of the center. The rule is frequently given that the camber should be *one four-hundredth of the radius;* but this is too great for the excessively heavy centers ordinarily used. It is probably better to build the centers true, and guard against undue settling by giving the frames great stiffness; and then if unexpected settling does take place, tighten the striking wedges slightly.

The two sides of the arch should be carried up equally fast, to prevent distortion of the center.

751. EXAMPLES OF ACTUAL CENTERS. For an example of a center employed in a tunnel, see Fig. 147 (page 512).

Fig. 158 (page 524) shows the center designed for the 60-foot granite arches of the recently completed Washington bridge over the Harlem River, New York City.* The bridge is 80 feet wide, and fifteen ribs were employed. Notice that the center does not extend to the springing line of the arch; the first fifteen feet of the arch were laid by a template.

Fig. 159 † (page 525) shows the center employed in constructing the Cabin John arch, which carries the Washington (D. C.) aqueduct over a creek, and which is the largest masonry arch in the world (see No. 2, Table 63, page 502). The arch is 20 feet wide, and five ribs were employed.

752. STRIKING THE CENTER. The Method. The ends of the ribs or center-frames usually rest upon a timber lying parallel to, and near, the springing line of the arch. This timber is supported by wedges, preferably of hard wood, resting upon a second stick, which is in turn supported by wooden posts—usually one under each end of each rib. The wedges between the two timbers, as above, are used

* Published by permission of Wm. R. Hutton, chief engineer.

† Compiled from photographs taken during the progress of the work (1856–60), by courtesy of Gen. M. C. Meigs, chief engineer.

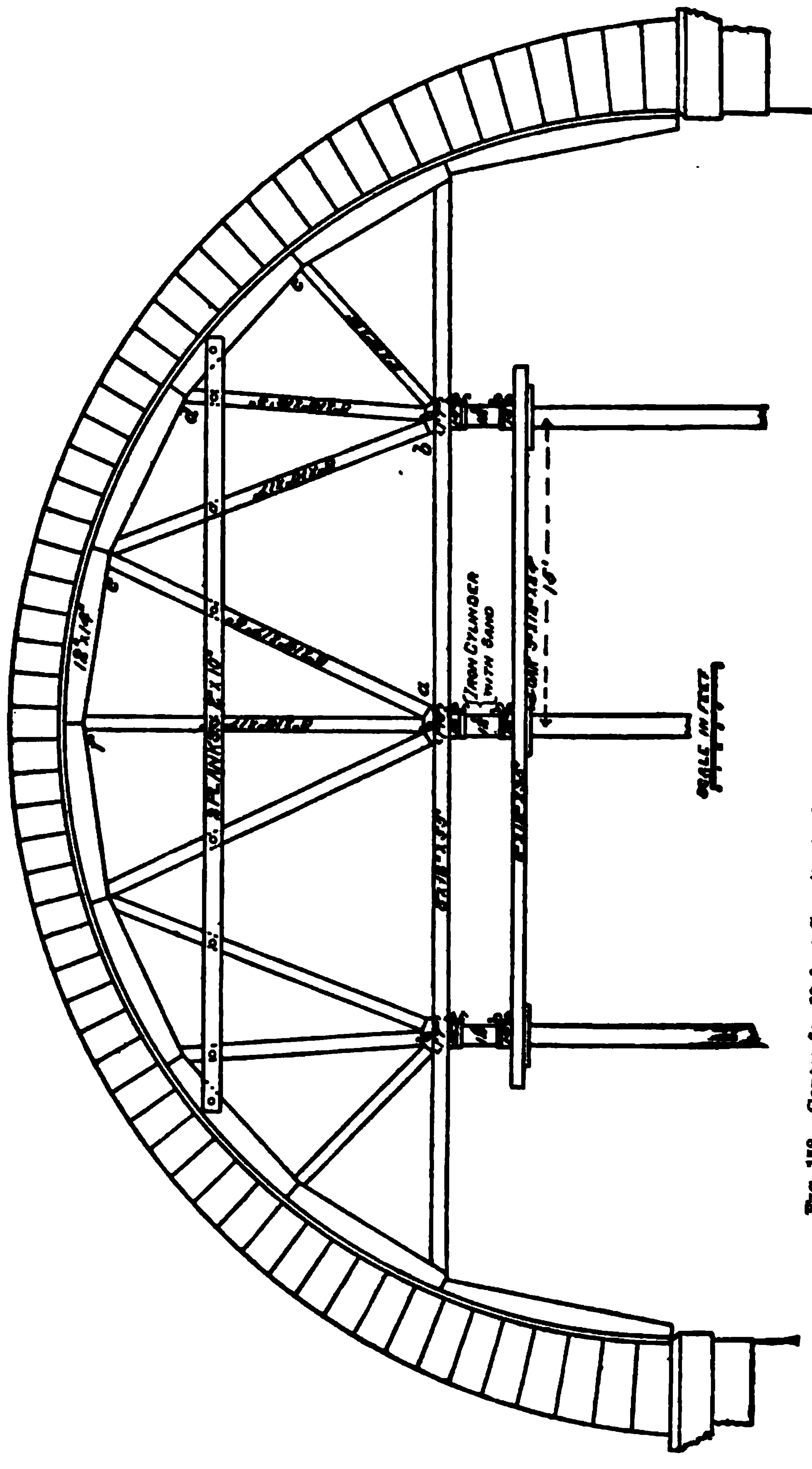

FIG. 158.—Center for 60-foot Granite Arches of Washington Bridge, Harlem River, New York City.

FIG. 159.—Cabin John Arch and Center, Washington (D. C.) Aqueduct.—For explanatory text see § 734 and § 751 (pages 501 and 502).

in removing the center after the arch is completed, and are known as *striking wedges.* They consist of a pair of folding wedges—1 to 2 feet long, 6 inches wide, and having a slope of from 1 to 5 to 1 to 10—placed under each end of each rib. It is necessary to remove the centers slowly, particularly for large arches; and hence the striking wedges should have a very slight taper,—the larger the span the smaller the taper.

The center is lowered by driving back the wedges. To lower the center uniformly, the wedges must be driven back equally. This is most easily accomplished by making a mark on the side of each pair of wedges before commencing to drive, and then moving each the same amount.

753. Instead of separate pairs of folding wedges, as above, a compound wedge, Fig. 160, is sometimes employed. The pieces

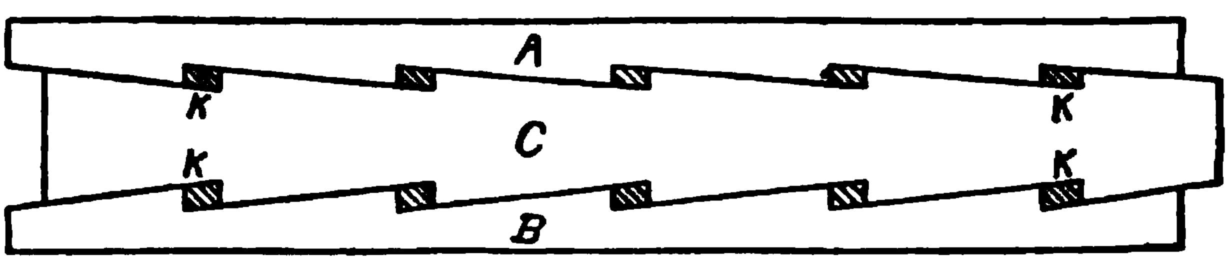

FIG. 160.

A and *B* are termed striking plates. The ribs rest upon the former, and the latter is supported by the wooden posts before referred to. The wedge *C* is held in place during the construction of the arch by the keys, *K, K,* etc., each of which is a pair of folding wedges. To lower the center, the keys are knocked out and the wedge *C* is driven back.

The piece *C* is usually as long as the arch, and supports one end of all the ribs; but with large arches, say 80 to 100 feet span, it is customary to support each rib on a compound wedge running parallel to the chord of the center (perpendicular to the axis of the arch). Instead of cutting the striking plates *A* and *B* as shown in Fig. 160, the compound wedge may play between tapered blocks gained into *A* and *B*. The piece *C* is usually made of an oak stick 10 or 12 inches square. The individual wedges are from 4 to 6 feet long.

For the larger arches, the compound wedge is driven back with a heavy log battering-ram suspended by ropes and swung back and forth by hand. The inclined surfaces of the wedges

should be lubricated when the center is set up, so as to facilitate the striking.

754. An ingenious device, first employed at the Pont d'Alma arch—141 feet span and 28 feet rise,—consisted in supporting the center-frames by wooden pistons or plungers, the feet of which rested on sand confined in plate-iron cylinders 1 foot in diameter and about 1 foot high. Near the bottom of each cylinder there was a plug which could be withdrawn and replaced at pleasure, by means of which the outflow of the sand was regulated, and consequently also the descent of the center. This method is particularly useful for large arches, owing to the greater facility with which the center can be lowered. See Fig. 158, page 524.

755. The Time. There is a great difference of opinion as to the proper time for striking centers. Some hold that the center should be struck as soon as the arch is completed and the spandrel filling is in place; while others contend that the mortar should be given time to harden. It is probably best to slacken the centers as soon as the keystone is in place, so as to bring all the joints under pressure. The length of time which should elapse before the centers are finally removed should vary with the kind of mortar employed (see Fig. 5, page 89) and also with its amount. In brick and rubble arches a large proportion of the arch ring consists of mortar; and if the center is removed too soon, the compression of this mortar might cause a serious or even dangerous deformation of the arch. Hence the centers of such arches should remain until the mortar has not only set, but has attained a considerable part of its ultimate strength (see Fig. 5, page 89),—this depending somewhat upon the maximum compression in the arch. It is probable that a knowledge of the elasticity and of the "set" of mortar would give some light as to the best time to strike centers; but unfortunately our information on those topics is very limited (see § 146).

Frequently the centers of bridge arches are not removed for three or four months after the arch is completed; but usually the centers for the arches of tunnels, sewers, and culverts are removed as soon as the arch is turned and, say, half of the spandrel filling is in place.

APPENDIX I.

SPECIFICATIONS FOR MASONRY.*

CONTENTS.

RAILROAD MASONRY.†

General Provisions. All stone used for the different classes of masonry must be furnished from the best quarries in the vicinity, subject to the approval of the engineer. Brick masonry shall at all times be substituted for stone, when so desired by the engineer.

Inspection. All materials will be subject to rigid inspection, and any that have been condemned must be immediately removed from the site of the work. The work will be done under the supervision of an inspector, whose duties will be to see that the requirements of these specifications are carried out; but his presence is in no way to be presumed to release in any degree the responsibility or obligation of the contractor.

Laying Masonry. All classes of masonry laid in cement must be neatly pointed with cement mortar, finely tempered. No masonry of any kind must be covered until it has been inspected and accepted by the engineer. No masonry will be allowed to be laid in freezing weather. [Many specifications omit this condition. See "Specifications for Laying Masonry in Freezing Weather," page 543.]

Measurement of Masonry. All masonry and brick-work will be built according to the plans and instructions furnished by the engineer, and will be estimated and paid for by the cubic yard, computing only the actual solidity thereof. No constructive or conventional measurement will be allowed, any rule or custom in the section of the country through which the road passes to the contrary notwithstanding. The price per cubic yard paid for masonry and brick-work will include the furnishing of all material, scaffolding, centering, and all other expenses necessary to the construction and completion of the masonry or brick-work. All "dressed" or "cut-stone" work—such as copings, bridge-seats, cornices, belt-courses, water-tables, brackets, corbels, etc.—furnished under the plans of the engineer will be paid for by the cubic yard, under the classification of the masonry in which they occur, with an additional price per square foot of the entire superficial surface of the stones "dressed," or "cut," or "bush-hammered."

Allowance for Extras. No allowance will be made for timber, or work on same, used in scaffolding, shoring, or centering for arches,—excepting only timber, sheet-piling, or foundation plank, necessarily, and by order of the engineer, left in the ground. No allowance will be made to the contractor for

* See also the specifications in the body of the book. See "Specifications" in Index.

† These specifications are the same, except in form, as those employed in the construction of the "West Shore" Railroad, but do not differ materially from those used in other roads, and have frequently been accepted as the standard.

any damage he may sustain by reason of floods or other causes; but such draining, bailing, or pumping from foundations as the engineer may decide to be necessary will be paid for at a price to be fixed by the engineer.

First-class Masonry will consist of quarry-faced ashlar [see §§ 200–07] laid in horizontal courses having parallel beds and vertical joints, of not less than ten inches (10″) nor more than thirty inches (30″) in thickness,—decreasing in thickness regularly from the bottom to the top of the wall,—laid flush in cement mortar of the quality hereinafter specified. Each course must be thoroughly grouted before the succeeding one is laid.

Size of Stones. *Stretchers* must be not less than two and one half feet (2½′) nor more than six feet (6′) in length, and not less than one and one half feet (1½′) in width, nor less in width than one and one half (1½) times their depth. *Headers* must not be less than three and one half feet (3½′) nor more than four and one half feet (4½′) in length—where the thickness of the wall will admit of the same,—and not less than one and one half feet (1½′) in width, nor less in width than they are in depth of course.

Cutting. Every stone must be laid on its natural bed. All stones must have their beds well dressed, parallel and true to the proper line, and made always as large as the stone will admit of. The beds and sides of the stone must be cut, before being placed on the work, so as to form joints not exceeding one half inch (½″) in width. No hammering on a stone will be allowed after it is set; but if any inequalities occur, they must be pointed off. The vertical joints of the face must be not less than eight inches (8″) in from the face, and as much more as the stone will admit of. All corners and batter lines must be run with a neat chisel draft one and one half inches (1½″) on each face. The projections of the quarry face beyond the draft lines must not exceed four inches (4″); and in the side-walls of tunnels this projection must not exceed two inches (2″).

Bond. The masonry shall consist of headers and stretchers alternating. At least one fourth of it shall consist of headers extending entirely through the wall, and every header shall be immediately over a stretcher of the underlying course. The stones of each course shall be so arranged as to form a proper bond—in no case less than one foot (1′)—with the stones of the underlying course.

Backing. The backing shall be of good-sized, well-shaped stones, laid so as to break joints and thoroughly bond the work in all directions, and leave no spaces between them over six inches (6″) in width, which spaces shall be filled with small stones and spalls well grouted.

Coping. All bridge-seats and tops of walls will be finished with a coping course of such dimensions and projections as may be ordered by the engineer, dressed and cut to a true surface on top and front edges, in conformity with diagrams for same which will be furnished by the engineer.

Foundation Courses. All foundation courses must be laid with selected large flat stones, not less than twelve inches (12″) thick, nor of less superficial surface than fifteen (15) square feet.

Second-class Masonry [§§ 208–12] will consist of broken range rubble of superior quality, laid with horizontal beds and vertical joints on the face, with no stone less than eight inches (8″) in thickness—unless otherwise directed by the engineer,—well bonded, and leveled as well as can be without hammer-dressing. No mortar joint shall exceed three quarters of an inch (¾″) in thickness. All corners and quoins shall have hammer-dressed beds and joints; and all corners and batter lines shall be run with an inch-and-one-half (1½″) chisel draft. At least one fourth (¼) of the stones in the face must be headers evenly distributed through the wall.

Bridge-seats and tops of walls shall be coped in the same manner as specified for first-class masonry. Stones in foundation courses shall be not less than twelve inches (12″) thick, and shall contain not less than twelve (12) square feet of surface.

Third-class Masonry will consist of good substantial rubble [§§ 213–17] laid in cement mortar. All stones shall be perfectly sound, and sufficiently large to make good, well-bonded, strong work; and shall be laid on their natural beds, in the most substantial manner, and with as much neatness as this description of work admits of. The stones in the foundations must be not less than ten inches (10″) thick, and shall contain not less than ten (10) square feet of surface; and each shall be firmly, solidly, and carefully laid.

First-class Arch-culvert Masonry shall be built in accordance with the specifications for first-class masonry, with the exception of the arch sheeting and the ring-stones. The rings shall be dressed to such size and shape as the engineer shall direct. The ring-stones and sheeting-stones shall not be of less thickness than ten inches (10″) on the intrados, and shall be dressed with three eighths inch (⅜″) joints, and shall be of the full depth specified (by drawings or otherwise) for the thickness of the arch. The joints must be made on truly radial lines, and the face of the sheeting-stones must be dressed to make close joints with the center. The ring-stone and sheeting-stones shall break joints by not less than one foot (1′).

The wing walls shall be neatly stepped, in accordance with the drawings furnished, with selected stones of the full width of the wing and of not less than ten inches (10″) in thickness, no stone being covered less than eighteen inches (18″) by the one next above it; or the wing shall be finished with a neatly capped newel at the end, and a coping course,—as may be selected by the engineer. The parapet shall be finished with a coping course of full width of parapet, with such projection as may be directed by the engineer, the stone to be not less than ten inches (10″) thick.

Second-class Arch-culvert Masonry shall be of the same general character and description as second-class masonry, with the exception of the ring-stones and the arch sheeting. The former shall be dressed as specified for first-class arch-culvert masonry. The latter shall consist of selected stones of the full depth of the arch, and shall have a good bearing throughout the thickness of the arch, and shall be well bonded. No stone shall be less than six inches (6″) in thickness on the intrados.

Box-culvert Masonry will be good rubble [see §§ 213–17], neatly laid up with square-shaped stones of a size and quality satisfactory to the engineer. The end parapet walls and also the side walls for three feet (3′) from the ends shall be laid in good cement mortar. When box culverts are ordered to be laid up entirely in cement mortar [see § 214], they will be classified as third-class masonry, and must conform to the specifications for the same.

The covering-stone for all box culverts shall be not less than ten inches (10″) in thickness, and must have a good, solid, well-leveled bearing on the side walls of not less than fifteen inches (15″).

Vitrified Pipe. In localities where but a small quantity of water passes, vitrified pipe will be substituted for culverts when so ordered by the engineer. Sizes of twelve (12″), fifteen (15″), or eighteen (18″) inches in diameter may be used, and must be of the best quality double strength, vitrified culvert pipe, subject to the approval of the engineer. Vitrified pipes must be well and carefully bedded and laid [see Figs. 97–99, pages 409–10], in accordance with the instructions of the engineer.

Retaining Walls will be classified as second- or third-class masonry, laid dry, as may be ordered in each particular case.

Slope Walls will be of such thickness and slope as directed by the engineer. The stones must reach entirely through the wall, and be not less than four inches (4″) thick and twelve inches (12″) long, laid with close joints, and as free as possible from spalls. The foundations must be prepared and laid as directed by the engineer.

Stone Paving shall be made by setting on edge stone from eight (8″) to

fifteen inches (15″) in depth, laid either dry or grouted with strong cement mortar, as may be directed by the engineer.

Riprap. When required by the engineer, the face of embankments and the foot of slopes shall be protected from the action of water by a facing of riprap stone, or of brush and stones, or by a retaining wall, as may be directed. The riprap, when used, shall be laid by hand by competent workmen, and shall be of such thickness and slope and of such undressed stone as the engineer may direct

Brick Masonry. The brick must be of the best quality [see § 57], well tempered, hard burned, and $8\frac{1}{4} \times 4 \times 2\frac{1}{4}$ inches.* No bats, cracked, crooked, or salmon bricks will under any circumstances be allowed in the work. The brick shall be well soaked in water before being laid, and shall be laid in hydraulic cement mortar of the quality hereafter specified, with such thickness of joint and style of bond [§ 242 and § 733] as shall be prescribed by the engineer. Grout will be substituted for mortar when ordered by the engineer.

Brick arching must be covered on the back with a coat of strong cement mortar one inch (1″) thick. In tunnel arching wherever a seam of water is met, the arch must be covered with roofing felt; or with a course of asphaltum (applied hot) of such thickness as may be directed by the engineer, and this covered with a plastering of cement mortar so as to make the arch impervious to water. A properly formed drainage channel shall be left in the backing of the arch and side walls, with suitable openings for the escape of the water, at such points and of such size as may be directed by the engineer. The keying of all arches shall be most carefully done, and in such manner as may from time to time be directed by the engineer. The packing between the arch and tunnel roof shall never be put in until at least forty-eight (48) hours after the section has been keyed.

Cement. The cement must be of the best quality of freshly ground hydraulic cement [of the Rosendale type—see § 72], and be equal in quality to the best brands of . . . . . . cement. It will be subject to test by the engineer or his appointed inspector, and must stand a tensile stress of fifty (50) pounds per square inch of sectional area on specimens allowed a set of thirty (30) minutes in air and twenty-four (24) hours under water [see § 90, and art. 5 of Chapter III].

Mortar. The mortar shall in all cases be composed of one (1) part in bulk of the above specified hydraulic cement to two (2) parts in bulk of clean, sharp sand, well and thoroughly mixed together in a clean box of boards, before the addition of the water. It must be used immediately after being mixed; and no mortar left over night will, under any pretext, be allowed to be used. The sand and cement used will at all times be subject to inspection, test, and acceptance or rejection by the engineer.

Concrete. The concrete shall be composed of two (2) parts in bulk of hard, sound, and acceptable stone—broken to a size that will pass in any direction through a two-inch (2″) ring, thoroughly clean and free from mud, dust, dirt, or any earthy admixture whatever,—and one (1) part of freshly-made cement mortar of the quality above described. The concrete shall be quickly laid in sections, in layers not exceeding nine (9) inches in thickness, and shall be thoroughly rammed until the water flushes to the surface. It shall be allowed at least twelve (12) hours to set before any work is laid on it.

Foundations. **Excavations.** Foundations for masonry shall be excavated to such depths as may be necessary to secure a solid bearing for the masonry, —of which the engineer shall be the judge. The materials excavated will be

* Instead of the dimensions as above, the specifications of which these are a revision and also many others contain the term "standard size," but until recently that term could have had no special significance (see § 62, page 46).

classified and paid for, as provided for in the Specifications for Grading. All materials taken from the excavations for foundations, if of proper quality, shall be deposited in the contiguous embankment; and any material unfit for such purpose shall be deposited outside the roadway, or in such place as the engineer shall direct, and so that it shall not interfere with any drain or water course. In case of foundations in rock, the rock must be excavated to such depth and in such form as may be required by the engineer, and must be dressed level to receive the foundation course.

Artificial Foundations. When a safe and solid foundation for the masonry can not be found at a reasonable depth (of which the engineer is to be the judge), the contractor shall prepare such artificial foundations as the engineer may direct.

Paving. Box culverts and small bridge abutments may have a paved foundation, if so directed by the engineer, by setting stones on edge, breaking joints, and extending across the entire width of the foundation.

Timber. Timber foundations shall be such as the engineer may by drawings or otherwise prescribe, and will be paid for by the thousand feet, board measure,—the price to include the cost of material, framing, and putting in place. All timber must be sound, straight-grained, and free from sap, loose or rotten knots, wind shakes, or any other defect that would impair its strength or durability. It must be sawed (or hewed) perfectly straight and to exact dimensions, with full corners and square edges. All framing must be done in a thorough, workmanlike manner. Both material and workmanship will be subject to the inspection and acceptance of the engineer.

Piling. All piles shall be of young, sound, and thrifty white oak, yellow pine or other timber equally good for the purpose, acceptable to the engineer. They must be at least eight inches (8") in diameter at the small end and twelve inches (12") in diameter at the butt when sawn off; and must be perfectly straight and be trimmed close, and have the bark stripped off before they are driven. The piles must be driven into hard bottom until they do not move more than one half inch (½") under the blow of a hammer weighing two thousand (2,000) pounds, falling twenty-five feet (25') at the last blow. They must be driven vertically and at the distances apart, transversely and longitudinally, required by the plans or directions of the engineer. They must be cut off square at the butt and be well sharpened to a point; and when necessary, in the opinion of the engineer, shall be shod with iron and the heads bound with iron hoops of such dimensions as he may direct,—which will be paid for the same as other iron-work used in foundations.

The necessary length of piles shall be ascertained by driving test piles in different parts of the localities in which they are to be used. In case a single pile shall not prove long enough to reach hard bottom, two shall be spliced together as follows: The head shall be sawed off square, and a hole two inches (2") in diameter and twelve inches (12") deep shall be bored into it; and into this hole a circular white oak treenail twenty-three inches (23") in length shall be well driven. Then another pile similarly squared and bored, and of as large a diameter at the small end as can be procured, shall be placed upon the lower pile, brought to its proper position, and driven as before directed. All piles, when driven to the required depth, are to be cut off truly square and horizontal at the height given by the engineer; and only the actual number of lineal feet of the piles left for use in the foundations after being sawed off, will be paid for.

Iron. All wrought and cast-iron work ordered by the engineer will be paid for by the pound,—the price to include the cost of material, manufacture, and placing in the work.

Coffer-dams. Where coffer-dams are, in the opinion of the engineer, required for foundations, the prices provided in the contract for timber, piles, and iron in foundations, will be allowed for the material and work on same,

which is understood as covering all risks from high water or otherwise, draining, bailing, pumping, and all materials connected with the coffer-dams. Sheet-piling will be classed as plank in foundations; and if left in the ground will be paid for by the thousand feet (1,000'), board measure.

Railroad Buildings.*

Tools. All tools necessary for the execution of the contract, including mortar boxes, will be furnished by the contractor at his own expense.

Staging. All staging required for the execution of the work done under contract shall be furnished by the contractor at his own expense. The railway company will, however, upon the completion of any structure, purchase of the contractor such staging material as it can advantageously use, and pay the contractor for such material an amount which, in the opinion of the railway company's engineer, shall seem reasonable and just.

Excavations. Dry excavations, or excavations above water, will be made by the contractor when so ordered by the railway company. Wet excavations, or excavations below water, will be made by the railway company, excepting when a special arrangement is made with the contractor. All excavations will be classified as either earth, loose rock, or solid rock.

When the excavation for any structure is made entirely by the contractor, it shall be measured in cubic yards, and paid for at the price per cubic yard specified in the contract. When an excavation is made in part by the railway company's force and is finished by the contractor's force, or when contractor's force assists railway company's force in making any excavation, contractor will be paid for the actual time that his force is employed, at laborer's current rate per day plus ten per cent. In case contractor furnishes a foreman for such work, time charged for foreman must not exceed one day for foreman for each ten days of labor, and contractor will be paid for the services of such foreman at a rate per day not to exceed the current wages paid foremen of laborers plus ten per cent. In case contractor uses masons, foremen of masons, or other skilled labor for the execution of the above "extra" or "time" work, the wages and time allowed will be the same as it would be if the work had been performed and supervised by laborers and foremen of laborers. When "extra" or "time" work is performed by contractor's force, and is supervised by contractor's foreman, who at the same time and place has charge of and is supervising "contract" work, no pay will be allowed contractor for such supervision, except when, in the opinion of the railway company's engineer, it may seem reasonable and just.

All excavations shall be made strictly in accordance with the plans furnished by the railway company and the stakes set by the railway company's engineer, and shall be executed in a neat and workmanlike manner. Where excavations are made under the supervision of the contractor, his agent or foreman, any erroneous or unnecessary excavation, and any masonry consequent to such erroneous or unnecessary excavation, shall be entirely at the contractor's expense, unless the contractor can show that such unnecessary work was caused by errors in the plans furnished by the railway company, or by errors in the railway company's engineer's stakes or instructions.

When excavation is made for concrete, great care must be taken to make the pits or trenches, as the case may be, of the exact width and depth required for the concrete, and any unnecessary excavation made or concrete used on account of lack of such care on the part of the contractor will be at his expense. Excavations for stone footing courses will be made, when not other-

* Atchison, Topeka and Santa Fé Railroad.

wise ordered, eight inches (8") (four inches (4") on each side) wider than the footing course. Excavations for walls not having footing courses will be made, when not otherwise ordered, twelve inches (12") (six inches (6") on each side) wider than the wall is thick.

Before masonry is built, excavations must be cleared of all loose earth, mud, or other objectionable material.

Stone. Stone will be furnished by the contractor at his own expense, and be of a quality suitable for the different classes of masonry hereinafter specified, and be subject to the inspection and acceptance of the railway company's engineer. Stone will be loaded on cars and unloaded by the contractor at his own expense. Stone will be delivered by the railway company on the nearest available side track to the work, and no charges whatsoever will be allowed contractor for hauling stone from cars to the work, except in extreme cases, where, in the opinion of the railway company's engineer, such charges may appear reasonable and just.

Sand. All sand for mortar or concrete will be furnished by the contractor at his own expense. When, in the opinion of the railway company's engineer, sand can not be secured by contractor within reasonable distance by wagon haul and at a reasonable price, transportation by rail will be furnished by the railway company, it being optional with the railway company at what point sand shall be procured. When railway company furnishes transportation for sand, cars shall be loaded and unloaded by contractor at his own expense.

All sand furnished by contractor shall be clean and sharp, and subject to the inspection of, and rejection by, the railway company's engineer. When, in the opinion of the railway company's engineer, sand requires screening, it shall be screened by the contractor at his own expense.

Cement and Lime. All cement and lime will be furnished by the railway company at its own expense; and will be delivered on cars on the nearest available side track to the work. It shall be unloaded by the contractor at his own expense, and shall be piled up in such manner by him as the railway company's engineer may direct. Cement and lime shall be covered and protected from the weather by the contractor at his own expense, in such manner as seems suitable to the railway company's engineer; and the contractor will be held responsible for the value of any cement damaged on account of unsuitable protection.

Water. Water required for all work done under contract shall be furnished by the contractor at his own expense. No charges made by contractor for hauling water will be allowed. When, in the opinion of the railway company's engineer, water can not be procured by the contractor within reasonable wagon haul, or at a reasonable expense, it will be furnished by the railway company.

Mortar. Except when otherwise ordered, all mortar shall be thoroughly mixed in a box, in the following proportions: One (1) part cement, two (2) parts sand, with sufficient water to render the mixture of the proper consistency. Care must be taken to thoroughly mix the sand and cement dry, in the proportions specified, before the introduction of water into the mixture. Mortar shall not be mixed except as it is used, and no mortar must be allowed to stand over night in mortar boxes or elsewhere.

Concrete. All concrete shall consist of one (1) part cement, two (2) parts sand, and six (6) parts broken stone, together with sufficient water to mix the sand and cement to the consistency of good mortar for masonry. The proportion of sand, cement, broken stone, and the quantity of water used in the mixture, may be varied at the option of the railway company's engineer.

Stone shall be of a quality acceptable to the railway company's engineer, and be broken so that seventy-five (75) per cent. will pass through a two-inch (2") ring and so that all will pass through a two and one half inch ($2\frac{1}{2}$") ring. Broken stone shall be free from mud, dirt, and other objectionable

material, and shall be subject to the inspection of, and rejection by, the railway company's engineer.

The sand and cement must be thoroughly mixed dry, in a clean, tight mortar box, before the introduction of water; and after water is applied to the mixture, the whole must be worked over with hoes until a good mortar of proper consistency is secured. After the mortar is made, the broken stone must be thoroughly drenched with clean water, and then shall be added to the mixture in the proportion stated above—or in any other proportion which the railway company's engineer may specify. The concrete must then be worked over and mixed until each stone is completely covered with mortar and all spaces between the stones entirely filled with same.

The concrete shall be deposited in horizontal layers not exceeding twelve inches (12") in depth, and shall be thoroughly tamped when so required by the railway company's engineer.

Rubble Masonry. Rubble masonry will be classified as either heavy rubble, foundation rubble, pier rubble, or uncoursed hammer-squared rubble. The latter will be called for convenience squared rubble [see §§ 208–12].

Heavy Rubble. When not otherwise specified or shown on the plans, footing courses will be built of rubble masonry. When footing courses exceed thirty inches (30") in width, the masonry will be classified as heavy rubble; and when thirty inches (30") or less in width, the masonry will be classified as foundation rubble.

Heavy rubble footing courses shall be built of well-selected stone, which shall have a thickness not less than the height of the footing course. Each stone shall have a bottom bed of good surface over its entire area, which shall be horizontal when the stone is in position. As much of the upper surface of each stone as will be directly under the masonry to be put above the footing course shall be uniform and parallel to the bottom bed. At least one third ($\frac{1}{3}$) of the length of the footing course shall be built of through-stone, and a larger proportion shall be furnished by the contractor when, in the opinion of the railway company's engineer, more through-stone are required to secure stability. No stone shall be used which will not bond or extend under the masonry to be built above the footing course a distance equal to at least one third ($\frac{1}{3}$) the thickness or width of the masonry; and not more than two stones shall be used at any section to make up the total width of the footing course, and the exposed face of each stone shall be at least twelve inches (12") in length.

All stones must be roughly jointed with a hammer for a distance back from their faces equal to the projection or offset of the footing course. No spaces to exceed forty (40) square inches in area shall be filled with spalls or chips, and the total area of all spaces must not exceed five (5) per cent. of the area of the footing course.

All stone when placed in position must be thoroughly rammed until firmly embedded in a bed of mortar, which shall first be placed in bottom of excavation or trench, and after stone are placed in position, all joints must be well grouted with mortar. When so required by the railway company's engineer, footing courses shall be built exactly to the dimensions shown on drawings or specifications, or with their edges built to a line.

Foundation Rubble. In general, and when not otherwise specified, all masonry below the bottom of water table or below the top of rail for stone buildings, and all masonry below the sill of wooden buildings, will be classified as foundation rubble, except footing courses more than thirty inches (30") in width, which will be classified as heavy rubble. Foundation rubble may be required, however, for any portion or for all the masonry in any structure, in which case no additional price shall be allowed, except when, in the opinion of the railway company's engineer, it shall seem reasonable and just.

In this class of masonry no stone having an exposed face shall be less than one twenty-fourth ($\frac{1}{24}$) of a foot in cubical contents nor less than two inches (2") thick. Any stone smaller than this will be considered a spall;

and spalls are not to be used to exceed seven (7) per cent. of the entire mass. The contractor will not be required to furnish stone (except for through-stone) larger than one and one half feet (1½') in cubical contents, but the stone used shall not average less than one half (½) of a cubical foot in contents. No stone shall be used which does not bond, or extend into the wall, at least six inches (6"). One through-stone, whose face area shall not be less than one half (½) of a superficial foot, will be required for each sixteen (16) superficial feet of face measurement of wall, and more than this may be required by the railway company when, in the opinion of its engineer, a larger proportion of through-stone is required to secure stability; *provided*, however, that the contractor shall in no case be required to furnish through-stone to exceed ten (10) per cent. of the entire mass. At least twenty (20) per cent. of the entire masonry shall consist of headers, or bond stones. In walls twenty-four inches (24") thick or less, these headers shall be at least two thirds (⅔) the thickness of the wall in length; and in walls more than twenty-four inches (24") thick, they shall be of sufficient length and be so placed as, in the opinion of the railway company's engineer, seems necessary to secure well-bonded and stable work.

Each stone shall be laid in its quarry bed, and any stone set on edge, or with the planes of its stratification vertical, will be rejected and ordered removed at the expense of the contractor. Stones shall be firmly bedded in mortar, and all spaces and joints thoroughly grouted with same.

Pier Rubble. Piers or pedestals whose horizontal sectional area is nine (9) square feet or less will be classified as pier rubble. When this area exceeds nine (9) square feet, the masonry will be classed as foundation rubble. Footing courses for such piers, when not exceeding sixteen (16) square feet in area, will be classed as pier rubble; and when exceeding this area, they will be classified as heavy rubble.

Footing courses must be built, so far as practicable, in accordance with the preceding specifications for heavy rubble masonry. Masonry in piers above footing courses must be carefully built of well-selected stone, having horizontal beds and vertical joints, and be thoroughly bonded; corners and faces must be built true and plumb. The specifications for foundation rubble, so far as practicable, shall apply to this class of masonry.

Each pier or pedestal shall be furnished with a hammer-dressed cap-stone not less than six inches (6") thick, of same area as pier, which must be accurately set at the required level. The price of this cap-stone must be included in the contract price per cubic yard for this class of masonry.

Squared Rubble. When not otherwise specified, the walls of all stone buildings above the bottom of the water-table will be built of uncoursed squared rubble.

In general the specifications for foundation rubble will apply to this class of masonry, the difference between the two classes being in the construction and finish of the outside face. The outside face of the wall will be built of well-selected stones, as nearly uniform in color as possible, which shall be neatly squared to rectangular faces, and which in all cases shall be laid on their natural or quarry beds. The beds of the stones shall be horizontal and the side joints vertical, and no joints to exceed three fourths (¾) of an inch will be allowed. No stone having a face area of less than eighteen (18) square inches or a thickness less than three inches (3") shall be used; and the average face of all the stones shall not be less than seventy-two (72) square inches.

The inside face shall be built and finished in accordance with the specifications for foundation rubble.

Corners of all buildings shall be built up with quoin stones, uniform in size and arrangement, for which no extra pay will be allowed contractor. Drafts will be cut on the corners when so specified or shown on plans. All joints shall be cleaned or raked out for a distance of three quarters of an inch (¾"), and neatly pointed with a raised joint. The mortar used for pointing shall be composed of such material as the railway company's engineer may select.

Openings for windows, doors, or for other purposes, will be made in walls when specified or shown on plans. The jambs of such openings shall be neatly cut to a true and smooth surface, and be drove-tooled, crandalled, or tooth-axed [see pages 125–34, particularly 126 and 133], as may be required by the railway company's engineer. Bed-joints of jamb-stones must be carefully cut, so that no joint to exceed one half an inch (½") will appear on the exposed face of the jambs. Jamb-stones shall be uniform in height, and one half shall be through-stones. In general the arrangement of jamb-stones will be shown on drawings.

The contract price for any opening shall include the cost of cut-stone sills, lintels, arches, jamb-stones, or any other cut-stone work required for that opening. In case no contract price is made for any opening, the contractor will be paid such price as, in the opinion of the railway company's engineer, seems reasonable and just.

Cut stone shall be furnished and put in place by the contractor when so required by the railway company. The stone furnished shall be of the quality required for the work, and acceptable to the railway company's engineer; and must be cut strictly in accordance with the plans and specifications in each case, and must be so cut as to lie, when in position, on natural or quarry beds. Cut stone will be paid for at the price specified in contract, and in case cut stone is furnished by the contractor for which there is no contract price, a price will be paid which, in the opinion of the railway company's engineer, seems reasonable and just.

Cut stone, or dimension stone for cut-stone work, may be furnished by the railway company at its own expense, and the contractor required to set the cut stone in position, or to cut and set the rough dimension stone, in which case the contractor will be paid for the work either as "extra" or "time" work, or at a price which, in the opinion of the railway company's engineer, may seem reasonable and just.

Wall Masonry. All walls shall be built to a line both inside and outside, and both faces shall be finished with a smooth and uniform surface, which shall be flat-pointed with a trowel, in a neat and workmanlike manner.

The upper courses of all walls, when leveled or finished for the reception of superstructure, shall be provided with a through-stone at each end, and also one through-stone for at least each five (5) lineal feet of wall. These through-stone shall be dressed on their top beds and accurately set to a level one half inch (½") below the level of the bottom of the superstructure. Between these through-stone the walls must be carefully laid, with the upper beds of the stones brought up flush with the top of the above-described through-stones so as to secure a perfectly level surface for the top of the wall. In no case shall spalls or chips be used, except in vertical joints.

The contractor will make such openings in walls as are required for windows, doors, or other purposes. No additional pay will be allowed for such openings, except where jambs are to be cut, and cut-stone sills or lintels are required, in which case such price per opening will be allowed as, in the opinion of the railway company's engineer, may seem reasonable and just. Cut or dressed dimension-stone will be furnished and set in position when so required by plans or specifications, and will be paid for by the railway company at such price as may, in the opinion of its engineer, seem reasonable and just. Wood, iron, or other material which may be required to be built into the masonry shall be properly put into position by the contractor, and no extra pay shall be allowed for such work. The cubical contents of such material, however, will not be deducted from the measurement of the masonry.

When so required, the contractor shall plaster the outside surface of basement or other walls with hydraulic mortar, composed of such materials as the railway company may select, and for such work the railway company will pay the contractor a price per square yard in addition to the contract price of the masonry.

Foundations for Trestles. Foundations for trestle bents, such as are built for coal chutes, will be classified as foundation rubble, and must be built with great care. The lower footing course, when exceeding thirty inches (30") in width, will be classed as heavy rubble. The upper course shall have one hammer-dressed through-stone at each end of wall, and at least three such through-stones between the end through-stones; otherwise the top course will be finished in accordance with the second paragraph under "wall masonry" above. This does not apply to bent foundations inside of coal-chute building, which will be built in the same manner as foundation walls in general.

Well-wall Masonry. Well-walls will be classified as foundation rubble. Well masonry will be built under the supervision of the well foreman who has charge of the well excavation, and contractor's foreman shall execute the work strictly in accordance with instructions given by him. When well-walls are sunk, or settled, as the excavation is made great care must be taken to make the outside surface perfectly smooth and uniform; and as many headers, not to exceed the maximum heretofore specified, may be required as, in the opinion of the railway company's engineer or well foreman, are necessary to secure stability.

Measurement of Masonry. In measuring masonry paid for by the cubic yard, all openings will be deducted, and the number of cubic yards will be the actual cubical contents of the masonry built. The cubical contents of cut stone, iron work, timber or other material, built into the masonry by the contractor, will not be deducted from the cubical contents of the whole mass.

ARCHITECTURAL MASONRY.*

Permit. The contractor for the masonry shall take out a building permit, including water for himself and plasterer and all other contractors that may require water about the building during the progress of the work. This contractor shall also take out street and obstruction permit, and permit for building curb and retaining walls. The cost of the above permits is to be included in the estimate.

Grade. The inside grade at the building shall be such as the superintendent shall direct. At the time of starting any pier, this contractor shall ascertain from the superintendent the height the inside grade shall be set above the established outside grade, taking into consideration the settlement that may occur during the progress of the work.

Excavation. It is the intention that this contractor shall call at the building and examine for himself the exact situation of the building site. He shall remove from the premises all earth or débris, except that which the superintendent may consider good for use in the grading required about the building. This contractor shall complete such grading about the building as may be found necessary. All sidewalk stone that may be found in connection with the excavation shall be removed by the mason, the said stone becoming his property. The same shall apply to any foundation stone or other material that may be found in excavating, although none of said material shall be used in connection with the new work about the building.

This contractor shall excavate, according to drawings, for all walls, piers, areas, etc., the intention being that the general level shall be excavated simply to the level of the finished basement floor. All trenches shall be excavated to the neat size as near as practicable; and each shall be leveled to a line on the bottom, ready to receive the foundation. At such time as the superintendent shall

* Except in form, these specifications are those employed by Burnham & Root, architects, Chicago, for the Society of Savings Building, Cleveland, Ohio, and conform closely to the general form employed by these architects.

direct, this contractor shall level off the basement surfaces and floors of areas to a line finishing three inches (3") below the top of the level of the finished basement floors, and leave the surface ready to receive the work of other contractors. When considered necessary in the judgment of the superintendent, all earth shall be tamped solidly and then be wet.

If any pockets of quicksand are found, this contractor shall excavate the same, and fill in solidly with concrete composed of clean broken stone of a size that will pass through a two-inch (2") ring and English Portland cement, proportioned 1 to 3, rammed solidly into place in the pockets, in layers, as the superintendent may direct. None of the sand that may be found while excavating shall be used in connection with any of the work about the building.

After all foundations or retaining walls are in and fixed, this contractor shall tamp the earth solidly around them, leaving it level to a line within eighteen inches (18") of the finished grade, and ready to receive the work of other contractors.

Bailing. This contractor shall do all bailing and draining of trenches or basement surfaces that may be found necessary during the progress of the work.

Shoring. This contractor shall protect all walls of the adjoining buildings, underpin all walls that may be considered necessary—in the judgment of the superintendent—to place the new work or to prevent injury of the old work, make good all repairs, provide such cutting as may be found necessary to place the work, and leave the adjoining buildings as good as at the start. The cost of this work is to be included in his estimate. This contractor shall furnish and put in place any sheet piling that may be required to retain the earth while the footings are being put in, and include all costs of the same in his estimates.

Protection. This contractor shall use proper care and diligence in bracing and securing all parts of the work against storm, wind, and the action of frost. Every night during freezing weather, each pier or wall shall be covered on top with sail-cloth, and the covering shall extend down over the face of all green work.

Concrete Footings. This contractor shall provide a frame of the area of the pier, composed of two-inch (2") plank, so arranged that the parts can be withdrawn and the pier left isolated after the concrete is set [see § 300]. All footings not otherwise indicated shall be constructed of concrete furnished by this contractor. The cement shall be first-quality, fresh Utica, or any other equally good quality approved by the architects. The contractor at the time of submitting his proposal shall state the kind of cement he intends using. The sand shall be clean and sharp. The stone shall be clean limestone, crushed to a size that will pass through a two-inch (2") ring, and screened. The concrete shall be composed of these ingredients in the following proportions: one (1) part of hydraulic cement, one (1) part of sand, and two (2) parts of crushed limestone. The cement and sand shall be mixed dry, and the mixture wet with a quantity of water sufficient to reduce it to the consistency of mortar. The stone and mortar shall be thoroughly mixed and laid in trenches as soon as possible, in layers of not more than six inches (6") in thickness, and be rammed until the water rises freely to the top.

All concrete footings shall be carefully leveled or pitched with concrete, and be left ready to receive the piers, walls, or columns, in each case as particularly indicated on the drawings.

Railroad-Rail Footings. All railroad rails that may be required in connection with the foundations shall be of Bessemer steel, weighing not less than sixty-five (65) pounds per yard, straight and sound, cut to the neat lengths indicated on the drawings. All railroad rails shall be furnished by this contractor, and by him set in place to centers and levels as indicated on the diagrams. None of these railroad rails are to be painted.

The concrete used in connection with steel-rail footings shall be composed

of one (1) part of first-quality English Portland cement—or any other equally good quality approved by the architects,—one (1) part of clean sharp sand, and two (2) parts of clean limestone crushed to chestnut size. This concrete shall be mixed as for concrete footings, and shall be rammed in solidly between the rails; and each tier shall be neatly squared at the outer edge.

Rubble Masonry. All piers colored blue on the drawings shall be classed as cut stone, and shall be furnished and set in place by another contractor; but all walls colored blue on the drawings—referring particularly to foundation walls for boiler-house, foundation wall for staircase way in alley, area walls, curb walls, and curtain walls between piers—shall be classed as rubble masonry, and shall be furnished and set in place by the mason.

All stone used in connection with rubble masonry shall be of selected, large size, first-quality stone, laid to the lines on both sides, well fitted together and thoroughly pointed, frequent headers that extend through the wall being provided. All stone shall be not less than two feet six inches (2′ 6″) long, one foot six inches (1′ 6″) wide, and eight inches (8″) thick, except such as may be found necessary to level up a course to the required height. The intention is that all walls shall be laid in courses about one foot six inches (1′ 6″) in height, leveled off at each course. Each stone shall have hammer-dressed beds and joints, and shall be firmly bedded and be well cushioned into place. All joints shall be filled with mortar. The facing of all walls shall be laid random range, and the face of the stone shall be coarse bush-hammered.

At the time of completing the retaining walls, this contractor shall excavate at least one foot (1′) on the outside of the wall, and point up all joints on the outside; and then provide and apply a coat of first-quality English Portland cement, not less than a half inch (½″) thick, to the outside of the wall from top to bottom. No cement covering will be required on the curb walls. All joints showing inside the building shall be raked out and neatly pointed up with cement; and, in addition, the face of walls coming in connection with the area shall be squared up. the joints finishing not to exceed one half inch (½″) thick.

All curb walls that may be required to receive the side-walks shall be brought to such levels as the superintendent shall direct, and shall be cemented on top and left ready to receive the side-walks—which shall be furnished and set by another contractor. None of the screen walls shall be set in place until such time as the superintendent shall direct. The foundation for the staircase bay in the alley shall be set in place, after the building is partly completed, at such time as the superintendent may direct. This contractor, at the time of starting this work, shall furnish such anchors as may be considered necessary, in the judgment of the superintendent, to attach his work to that already in place, and shall do all cutting and fitting that may be found necessary to properly place his work.

Mortar for Rubble Masonry. All rubble masonry above referred to shall be laid in mortar composed of perfectly fresh Utica cement—or other equally as good approved by the architects,—mixed in the proportion of one (1) part of cement to two (2) parts of clean sharp coarse sand. The sand and cement shall be mixed in a box dry; then wet, tempered, and immediately used.

Common Brick-work. All walls or sections colored red on the drawings or otherwise indicated to be of brick, shall be of selected, first-quality, hard-burned Chicago sewer brick—or other equally good quality approved by the architects. The above quality of brick shall be used throughout the entire work, except that hollow fire-clay brick shall be used in connection with all curtains between windows on elevations above the first story, and for the backing of all stone-work above the top of the eighth-story floor-beams. No bats shall be used. No pressed or face brick will be required in connection with this work.

All brick shall be well wet, except in freezing weather, before being laid. Each brick shall be laid with a shove joint, in a full bed of mortar, all inter-

stices being thoroughly filled; and where the brick comes in connection with anchors, each one shall be "brought home" to do all the work possible. Up to and including the fifth story, every fourth course shall consist of a heading course of whole brick extending through the entire thickness of the walls; above the fifth story, every sixth course shall be a heading course. All mortar joints shall be neatly struck, as is customary for "first-class trowel work." All courses of brick-work shall be kept level, and the bonds shall be accurately preserved. When necessary to bring any course to the required height, clipped courses shall be formed, as in no case shall any mortar joints finish more than one half inch (½") thick. All brick-work shall be laid to the lines, and each tier kept plumb, the intention being that none of the window-frames shall be set in place until the roof is on.

All lintels over openings indicated in connection with brick partition walls in basement shall be of steel railroad rails, and shall be furnished and set in place by the mason. These rails shall be painted one coat of mineral paint before being brought to the building.

All cut stone shall be backed as fast as the superintendent may consider proper, and the mason shall build in all anchors that may be furnished by the contractor for the cut stone. When openings or slots are indicated in connection with walls, the size and position of the same shall be such as the superintendent shall direct, unless otherwise shown. This contractor shall leave openings to receive all registers that may be required in connection with the heating or ventilating system, and shall also leave openings in connection with the corner vaults at such places in the floor and ceiling as the superintendent shall direct.

All masonry that may be required at the time of setting the boilers shall be furnished and set in place by the contractor for steam-heating apparatus.

Mortar for Brick-work. All mortar used in connection with sewer brick, together with the mortar in the brick parapet walls and the chimney above the roof line, shall be composed of two (2) parts of lime mortar—made up very poor,—and one (1) part of first-quality Utica cement—or other equally good approved by the architects. Said mortar shall be used immediately after being mixed, and in no case shall any be used that has stood over night.

The remaining brick-work, including the fire-brick hereinafter referred to, shall be laid in mortar composed of best slaked lime and coarse sharp clean sand of approved quality.

Brick Arches. Where arches are indicated in connection with the first-story banking vault or in connection with roadway in the court on the north front of building, said arches shall be formed with common brick laid in rowlock courses, regularly bonded [see § 783]. The mortar for this work shall consist of one (1) part Portland cement and three (3) parts clean sharp sand. Each brick shall be laid with a shove joint; and each rowlock course shall be cemented on top at the time of laying the next course. The last course shall be cemented on top, and be left ready to receive the concrete floor or roadway —which shall be provided by another contractor.

All centers that may be required in connection with this work shall be furnished and set in place by the carpenter; and none of said centers shall be removed until such time as the superintendent shall direct. After the same have been removed, this contractor shall thoroughly clean down all face-work.

All iron indicated in connection with this work shall be furnished and set in place by the contractor for constructional iron work,—except the bearing plates, which shall be bedded by the mason.

Smoke Britching. The smoke britching indicated in connection with the main boiler-stack will be furnished and set in place by the contractor for constructional iron, although the mason shall back up the same at such time as the superintendent shall direct.

Fire-brick. The lining shown to stand alone in connection with the

boiler chimney in the lower stories shall be laid with first-quality fire brick, laid in stretcher courses, regularly bonded, with headers of whole b sixteen inches (16") apart in every sixth course to stay the linings, care be taken to preserve the air-space indicated.

All fire-clay brick shall be laid in first-class fire-clay mortar, each brick being laid with a solid joint neatly struck on each side with a trowel.

Hollow Fire-clay Brick. All brick used in connection with the spandrels above the first story on all elevations, together with all backing required in connection with the stone work above the top of the eighth-story floor-beams, shall consist of first-quality, hard-burned, fire-clay, hollow brick, equal in quality to sample to be seen at the office of the architects. Each brick shall be laid with a shove joint. This contractor shall point up this work, and leave the surfaces of the walls smooth and ready to receive plastering.

Cutting and Fitting. This contractor shall do, promptly and at the time the superintendent so directs, all cutting and fitting that may be required in connection with the mason-work by other contractors to make their work come right, and shall make good after them.

Setting Iron-work. It is the intention that all constructional iron-work shall be furnished and set in place by another contractor, and that all iron shall be hoisted from the outside of the building by means of a derrick. In setting the beams and columns in place, the mason shall keep pace with the contractor for constructional iron work, and at no time shall the mason be left one story behind the constructional iron-work. Each beam, girder, or column shown to rest on the masonry shall be provided with iron plates by the contractor for constructional iron, said plates being furnished to the mason at the sidewalk; and the mason shall set the same in place, firmly bedded in mortar, at such position or height as the superintendent shall direct.

All iron wall-plates that may be required to receive the fire-clay arches will be furnished at the sidewalk by the constructional-iron contractor; and this contractor shall set each in such position and at such height as the superintendent shall direct.

Cut Stone. All parts colored blue on the drawings, or otherwise indicated to be of stone, or usually classed as cut stone, shall be furnished and set in place by the contractor for cut stone. The same shall apply for the terra-cotta roof-copings indicated. All mortar, staging, or hoisting apparatus that may be required in connection with this work shall be furnished by the contractor for cut stone. All cut stone will be set from the outside; but the mason shall back up all cut-stone work in a manner approved by the superintendent.

Specifications for laying Masonry in Freezing Weather.

Only in case of absolute necessity will any masonry be laid in freezing weather [see § 144]. Any masonry laid in freezing weather must not be pointed until warm weather in the spring.

If necessary, masonry may be laid in freezing weather, *provided* the stone or brick are warmed sufficiently [§ 144] to remove ice from the surface, and the mortar is mixed with brine made as follows: "Dissolve one pound (1) of salt in eighteen gallons of water when the temperature is 32° F., and add one ounce of salt for every degree the temperature is below 30° F.—or enough salt, whatever the temperature, to prevent the mortar's freezing."

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